NUMERICAL MODELING AND ANALYSIS OF PILE SUPPORTED EMBANKMENTS

by

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ABSTRACT

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Very soft clays are often avoided in construction due to their low shear strength and high compressibility. Special construction methods are adopted when embankments are constructed on very soft clay or peat. Design of structures on soft ground, where the structure impose large loads onto the ground raises several concerns on factors like bearing capacity failure, differential settlements, lateral pressures and structural instability. Shallow and deep stabilization of soft ground are most frequently used special construction techniques adopted in geotechnical engineering.

Chemical (lime and cement) stabilization has been extensively used in both shallow and deep stabilization in order to improve inherent properties of soil such as strength and deformation. The shallow stabilization involves mixing surface soil with

stabilizers like lime. The other technique involves forming columns of chemically treated lime in the soft soil.

The lime column (chemico-pile) soil improvement method has been used in a full-scale test embankment project located at Nong Ngo Hao site near Bangkok, Thailand. A numerical solution was provided in order to evaluate the behavior of the embankment using Finite Element (FE) program PLAXIS. The predicted embankment behavior using PLAXIS has been compared with actual field data in terms of excess pore pressure, settlements, and lateral displacement. The predicated results were in good match with the actual field results. In addition, parametric studies were performed in order to understand the embankment behavior when reinforced with geosynthetics. Also, change in geosynthetic stiffness, pile stiffness, foundation soil modulus and embankment property were modeled. Results indicated that geosynthetics included in the embankment fill produced considerable increase in load transfer when placed at 100mm above ground surface.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Introduction

Embankments are among the most ancient form of civil engineering construction. Embankments are required for the construction of highways, runways, dams, and levees. Historically, in order to reduce the technical problems including compressibility and shear strength, and cost associated with their construction, embankments are preferred over sites with soil exhibiting good geotechnical characteristics. However, rapid growth and pavement construction in urban and costal areas have forced the use of weak sub grade sites for development of transportation related projects. Coastal regions are generally covered with soft compressible clayey deposits and mud. Embankments constructed on soft soils undergo large deformations and lateral movements which results in long construction delays and/or premature failure. Design of embankment on soft ground, where the structure impose a large load onto the ground raises several concerns on factors like bearing capacity failure, differential settlements, lateral pressures and instability. Stabilization of soft clay is one of the important construction techniques in geotechnical engineering. When embankments are constructed on very soft clay or peat, special construction methods are required.

Traditionally, geotechnical-engineering solutions to address the above-mentioned concerns include:

- consolidation of soft soil with vertical sand drains and preloading
- staged constructions
- using low weight fill materials like geofoams
- overexcavating of soft soil and replacement with suitable fill
- use of concrete raft foundation on pile systems
- soil reinforcement by geosynthetics

However, most of the above-mentioned special methods of constructions, consolidation, and staged construction may take considerable amount of time, low weight fill materials, overexcavating of soft soil and replacement with fill material, and use of concrete raft may be an expensive technique.

To overcome the time constraints and the cost issues, pile supported embankment with or without geosynthetic, reinforcement can be considered as an excellent engineering solution. This system consists installing piles up to the bearing stratum below the soft soils to support the embankment (Barchard, 1999), as shown in Figure 1.1.

Pile supported embankments are non-time dependent foundation technique used to increase the structural stability and to reduce the structural deformations (Ariema and Butler, 1990). In recent years, geosynthetics have been used in combination with pile or column system to support embankment over soft clay foundations (Han and Collin, 2005).

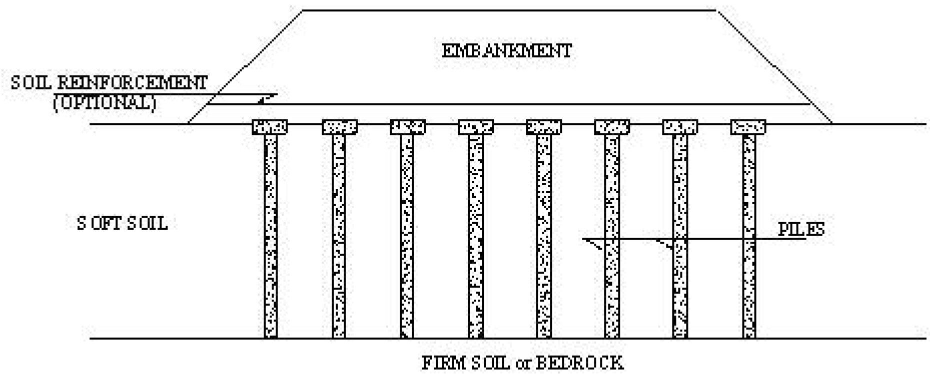


Figure 1.1 Pile Supported Embankment (Barchard, 1999)

The application of geosynthetic(s) in the fill just above the pile, enhances the load transfer efficiency, minimizes yielding of soil above pile, and reduces the total and differential settlement (Han and Gabr, 2002).

Pile supported embankment have been designed with or without geosynthetics reinforcement. The system without geosynthetic reinforcements is referred as conventional pile-supported embankment while the system with geosynthetic reinforcements is referred to as geosynthetic-reinforced pile supported embankment. Geosynthetic-reinforced embankment may be designed over piles with caps or on columnar systems. It is estimated that the pile covering as much as 10% of the area beneath the embankment may carry more than 60% of weight of the embankment due to arching action in the fill (Hewlett and Randolph, 1988). A single geosynthetic reinforcement layer behaves as a tensioned member while a multilayer system behaves like a stiffened platform (like a plate) which is due to the interlocking of geosynthetic reinforcement with the soil (Han and Gabr, 2002). These ground improvement-engineering techniques have been practiced for more than two decades (Han and Collin, 2005).

1.2 Benefits of Pile Supported Embankment

Several applications of pile-supported embankment (with or without geosynthetic reinforcement) have been reported over a period of time. As reported by Reid and Buchanan (1984), this technique was previously used to prevent the differential settlement at the approach embankment constructed over soft soil and a bridge abutment supported over a system of piles as presented in Figure 1.2. A chemico-lime pile instead of concrete pile was used to support the embankment over a soft clay deposit for an airport project in Bangkok, Thailand as illustrated in Figure 1.3. The reduction of surface settlement due to chemico-pile was more than 50% (Hossain and Rao, 2005).

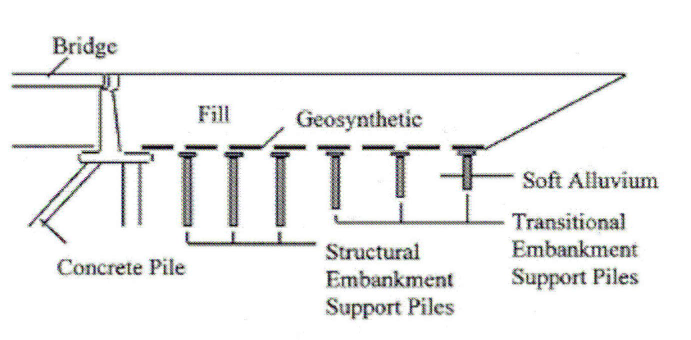


Figure 1.2 Bridge Approach Support Piling (Reid and Buchanan, 1984)

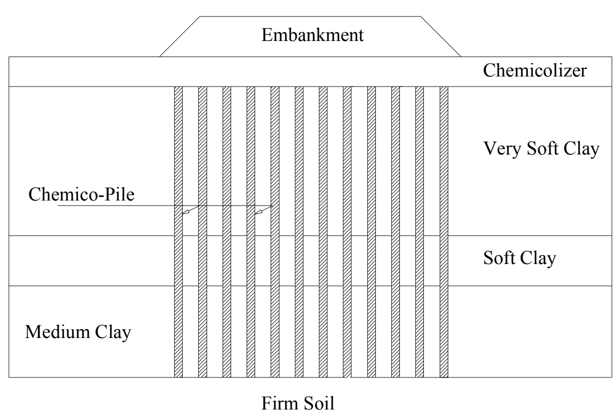


Figure 1.3 Embankment with Chemico-Pile (Hossain and Rao, 2005)

Deep soil-cement lime mixed columns in place of conventional concrete piles are also used to support embankment constructed over soft soil. The use of soil-cement mixed columns in combination with a layer of geogrid (Figure 1.4) under an embankment (pavement section) with a 11% coverage ratio was satisfactory in comparison with 50 – 70% coverage ratio (Figure 1.5) of pile caps according to Han (1975).

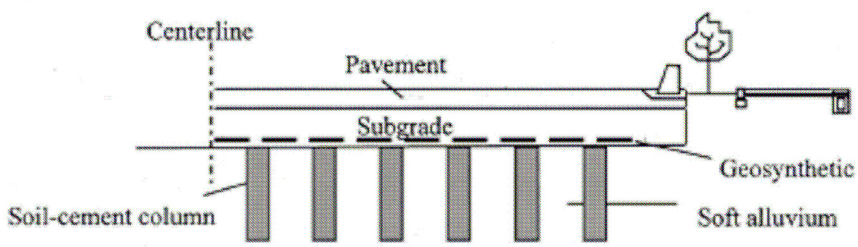


Figure 1.4 Subgrade Improvement (Han and Gabr, 2002)

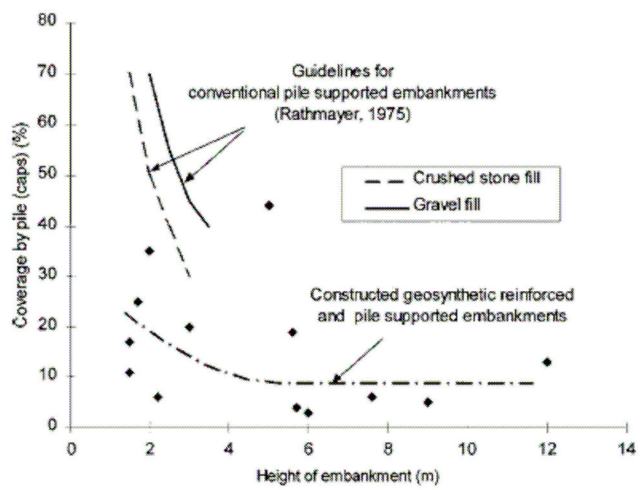


Figure 1.5 Coverage by Pile (Caps) for Construction Pile Supported Embankment (Han, 1975)

This technique with geosynthetic-reinforced earth platform in conjunction with vibroconcrete (Figure 1.6) is used under a storage tank to minimize total and differential settlement is a soft soils terrain.

This technique is most effective in supporting segmental retaining wall (Figure 1.7) as reported by Alzamora et al.,(2000).This technique is commonly used to prevent the differential settlement in case of widening the existing road embankment (Figure 1.8) over soft soil where the settlement has ceased over a period of time.

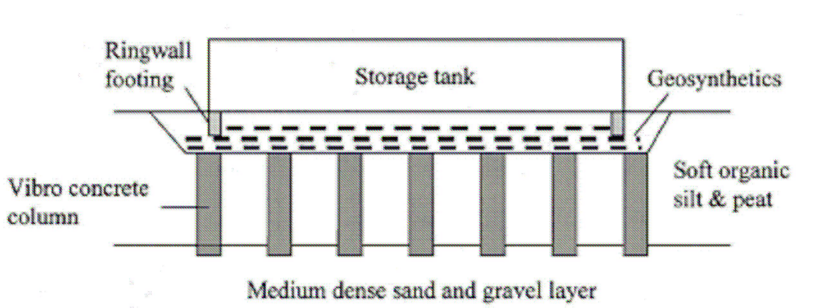


Figure 1.6 Storage Tank (Han and Gabr, 2002)

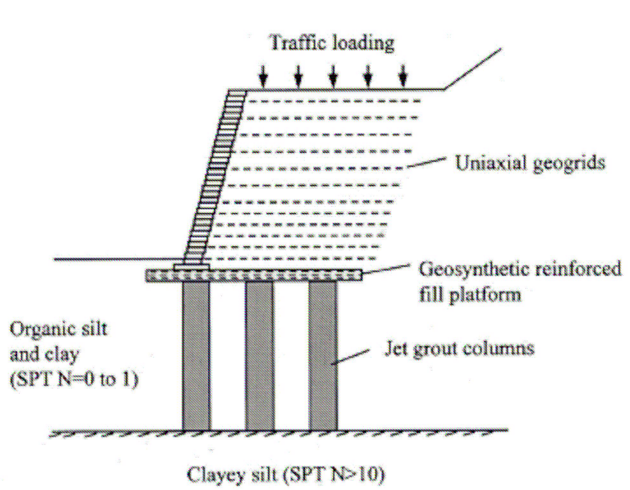


Figure 1.7 Segmental Retaining Walls (Alzamora et al., 2000)

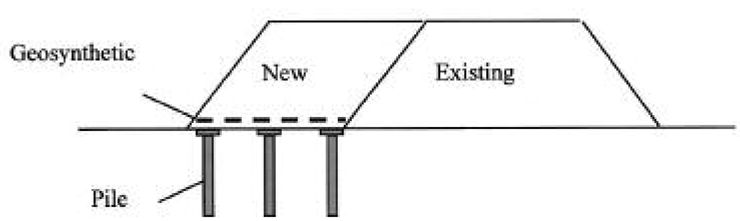


Figure 1.8 Widening of Existing Road (Han and Gabr, 2002)

Pile supported embankments have been used in geotechnical applications for more than 60 years, and the use of stone column technology was first implemented in Europe in the 1960's (AASHTO and FHWA, 2002). It is necessary to device a mathematical model that is capable of simulating the response to prescribed actions such that acceptable agreement between predicted results and observations of physical model can be obtained. The process of standardizing, modifying, and verifying mathematical model can take several forms. A common engineering practice is to construct a mathematical model and predict with its output. A numerical model may be calibrated against known results for a range of varying parameters. The calibration process can be systematized (also known as "system identification") if both the input and the output are known beforehand (Christian, 1987). On satisfactory agreement of the predicted output with the physical experiments confirms correctness of both mathematical model and the physical test.

1.3 Research Objective

While there are significant economic and operational advantages of construction of pile-supported embankment on soft clayey soil, a comprehensive study on different pile-supported embankment is limited. As such, there is a need to explain and study the design and modeling aspects of pile supported embankment. The main objective of the present research was:

(1) To conduct a comprehensive study, to identify and to investigate different types of pile-supported embankments.

(2) To model pile supported embankment using the available finite element modeling program PLAXIS.

(3) To analyze the effects of pile modulus, geosynthetic stiffness, soil modulus and height of embankment in pile supported embankment.

1.4 Research Report Organization

Chapter 1 - (Introduction) provides a brief insight to pile-supported embankment technology. It also reviews the common practices of stabilization techniques in conjunction with pile-supported embankment.

Chapter 2 - (Literature Reviews) gives a summary of the background of various techniques adopted for shallow and deep stabilization.

Chapter 3 - (Numerical Modeling) presents the fundamental numerical modeling concepts of pile-supported embankment.

Chapter 4 - (Numerical Modeling of Pile Supported Embankment) deals with the numerical modeling of pile supported embankment using PLAXIS and presents a case study for an embankment project in Bangkok, Thailand.

Chapter 5 - (Embankment Analysis and Design) presents the results and discussion of effects of different parameters on numerically modeled pile supported embankment.

Chapter 6 - summary and conclusions

CHAPTER 2

LITERATURE REVIEW

Very soft clays and peat soils are often avoided in construction due to their low shear strength, and high compressibility. When embankments are constructed on very soft clay or peat, special construction methods are required. Many ground improvement techniques like sand drains, wick drains, lightweight fills, and surface improvement techniques like applications of geosynthetics, surface soil treatment with lime are used to avoid excessive deformations.

Greenacre (1996), proposed a flowchart for construction on soft ground as presented in Figure 2.1. As reported by Greenacre (1996), one of the best alternative methods for constructing an embankment on soft soils is a piled supported embankment (Barchard, 1999). This alternative solution consists of placing the embankment over a grid of piles driven through highly compressively soil to a firm soils or the bedrock. Often, due to time constraints involved in construction and uncertainty of underlying soil conditions, the use of pile supported embankment is regarded as the most practical and economic option (Gomes, 1998). Embankments supported on pile system have been mainly to minimize the differential settlement in the embankment fills (Chew et al., 2004). The current design and analysis of pile-supported embankment ignores the contribution of compressible soft soil to support the weight of embankment fill. Piles are commonly designed to carry the full embankment load.

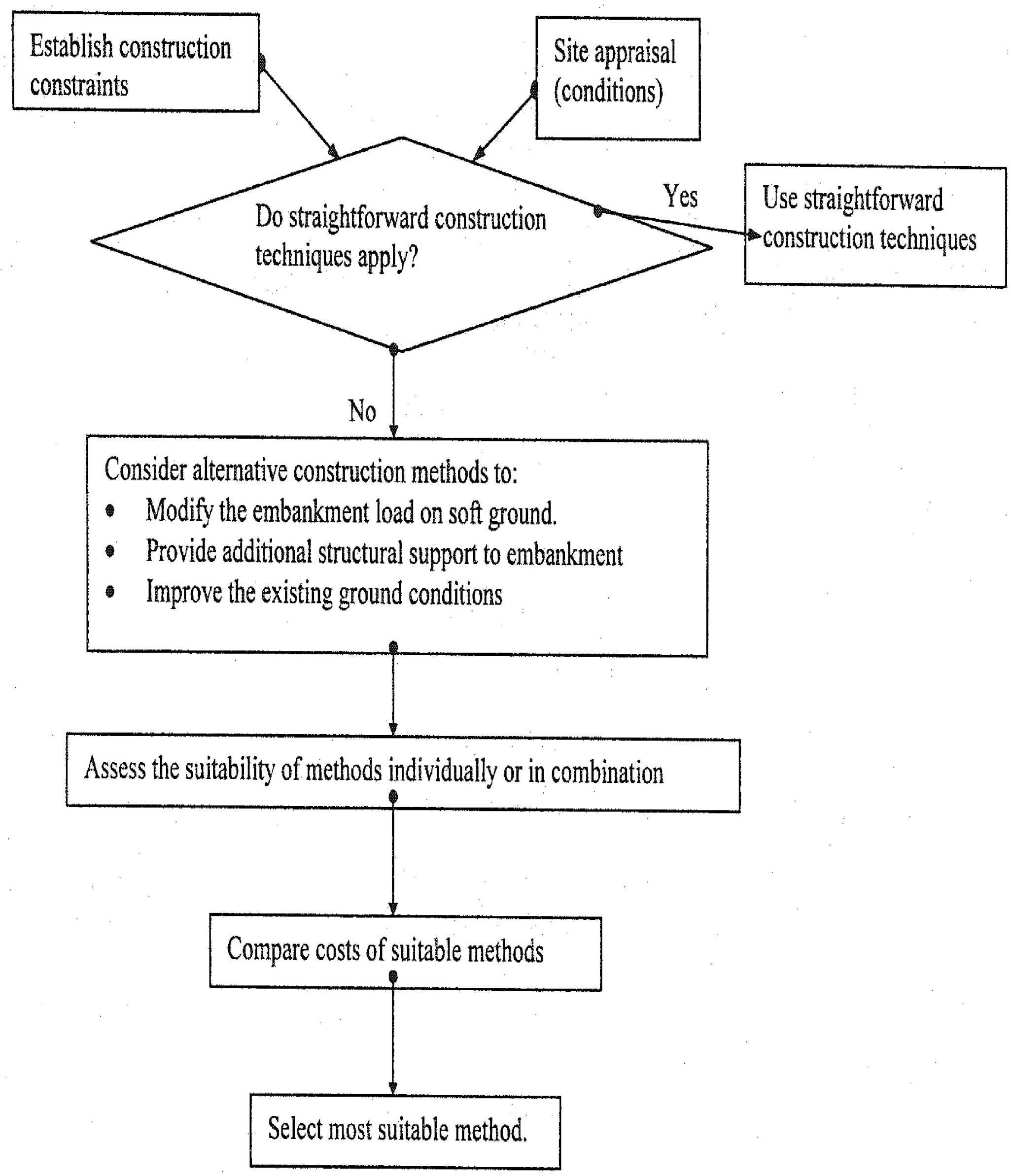


Figure 2.1 Option Flow Chart for Construction on Soft Ground
(Barchard, 1999)

Many of the technologies in current practice for embankment construction on soft soils offer potential cost saving in addition to improvement in the construction quality. In Europe, piled embankments are used to accelerate construction over classically method of surcharge with or without wick drains ("Innovative Technology for Accelerated Construction of Bridge and Embankment Foundation" by AASHTO and FHWA, 2002). An embankment mat (also called as Load Transfer Platform, LTP) support system may be required to transfer the load to the foundation soils or the pile system depending on the foundation soils conditions, type of pile system (which includes the geometry of piles, arrangement of piles, and spacing of piles), and loads from the embankment. At present geosynthetic reinforcement mat is used as LTP in conjunction with lightweight aggregates or geofoam. Stabilization of the upper 3 to 5 m of soil materials either by mass mixing or by rapid impact compaction would be an optimum foundation improvement technique which may be implemented with or with out deep foundation systems (AASHTO and FHWA, 2002). Some of the accelerated construction performances are presented in Table 2.1 and Table 2.2 for embankment mat system and embankment deep foundation system respectively.

Table 2.1 Embankment Mat Foundation System, Equipment for Acceleration Construction (AASHTO and FHWA, 2002)

Technology or Process	**Anticipated Accelerated Construction Performance**	**Comments**
Load Transfer Mat – Concrete Slab	No surcharge required; could use prefabricated mat	Soft foundation – higher cost
Load Transfer Mat – Concrete Caps	No surcharge required	Required stiff columns or piles that are closed spaced
Load Transfer Mat – Geosynthetics	No or reduced surcharge required	For very hard pile need to check for punching shear; works well for soft piles
Load Transfer Mat – Geosynthetics and caps	No or reduced surcharge required	Arching and spacing versus geosynthetic strength
Light Weight Aggregates	Reduces or eliminates surcharge	Application of geofoam
Mass Stabilization	Saves time as compared to preloading	Excellent for soft and organic clays
Automatic Controlled Variable Roller Compaction	Speed compaction eliminating wasted time	Compaction efficiency and uniformity improved

Table 2.2 Embankment Deep Foundation System, Equipment and Ground Improvement Methods for Acceleration Construction (AASHTO and FHWA, 2002)

Technology or Process	**Anticipated Accelerated Construction Performance**	**Comments**
Embankment on Piles	Saves surcharge time; no surcharge required	Newer piles (e.g., CFA, AU-Geo, Screw Piles) may reduce cost
Deep Mix – (Cement – Lime) Columns	Reduced surcharge required	Advancement in QC, mixing, equipment and uniformity
Mass Stabilization	Saves time when compared to preloading	Excellent for 3 – 5m depth in soft soils
Geotextile Encased Columns (GEC)	High bearing capacity, saves time require for surcharge	80% to 90% settlement in 3 months
Screw Piling	Similar to driven pile	Low capacity friction pile
Combination Soil Stabilization System	One step installation of cement column	Low weights, easy mobilized equipments
Continuous Flight Auger Piles	Rapid pile installation for vertical or slight batter piles	Installation rates 400 – 500m per day

2.1 Mechanisms of Load Transfer

Design of pile-supported embankment in conjunction with geosynthetic reinforcements are commonly adopted in present construction practice. The interaction between the piles, the soft foundation soil, embankment fill, and the geosynthetic reinforcement can be schematically described as shown in Figure 2.2. The embankments fill mass between the two consecutive pile caps has a tendency to move downwards due to the presence of soft soils under the fill. Shear resistance, τ, in fill mass above the pile cap restrain the movement of fill mass to some extent. The development of shear resistance (τ) reduces the pressure acting on the geosynthetic.

However, it increases the load applied onto the pile caps. The load transfer mechanism from the fill mass on to the pile caps is known as “Soil Arching Effects” (Han and Gabr, 2002). As summarized by McNulty (1965), arching is the ability of the material to transfer load from one point to another in response to relative displacement between the locations. Geosynthetic reinforcement enhances the load transfer from the fill soil to piles, reducing the total and differential settlements. The reduction of differential settlements at the base of the embankment is reflected at the surface of the embankment. Installation of geosynthetic reinforcement increases the load transfer and reduces the area replacement ratio of the columns (piles) as observed by Russell and Pierpoint (1997), Han and Wayne (2000), (Han and Gabr, 2002). Test conducted by Terzaghi (1936) and McNulty (1965) affirm that the shear stress induced by soil arching increases with the displacement and fill thickness above the yielding soil portion (Han and Collin, 2005).

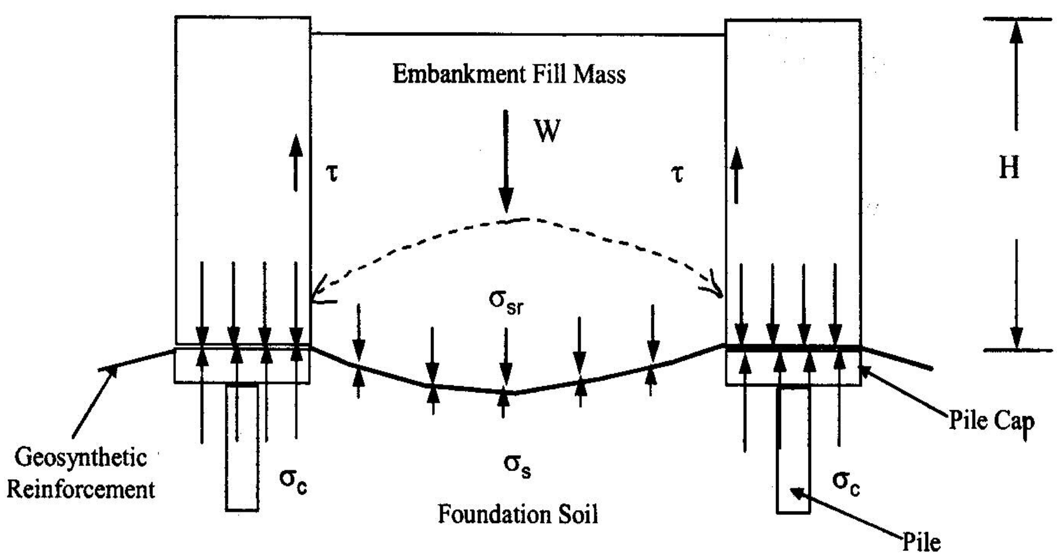


Figure 2.2 Load Transfer Mechanisms of Geosynthetic Reinforced
Pile-Supported Platform (Reinaldo and Ynog , 2003)

The degree of soil arching can be expressed as:

$$\rho = \frac{\sigma_{sr}}{\gamma H + q} \tag{2.1}$$

Where,

ρ : soil arching ratio

σ_{sr} : applied pressure on geosynthetics

γ : unit weight of embankment fill

H : height of fill

q : uniform surcharge on the embankment

The magnitude of soil arching ratio (*ρ*) range between 0 and 1. Complete soil arching is defined as 100% when "*ρ*" tends to zero. However, no soil arching is represented when "*ρ*" tends to unity.

The pile caps are designed to cover an adequate plan area of the embankment. The main objective of pile cap is to optimize arching in the fill and there by reducing the differential settlement. Circular pile caps are able to sustain more concentrated load than rectangular ones, which, can be illustrated by the concept of distribution of earth pressure over the pile caps. Reinforcing the embankment fill with geosynthetics further reduces settlements and enhances the load transfer to the pile in addition to increase in pile spacing. The coverage area of pile for most of the pile supported embankment construction ranges between 10-30% (Han and Gabr, 2002).

To summarize, load transfer is due to soil arching, tensioned member (or) stiffened platform effects and stress concentration (due to different stiffness between pile and soil). The magnitude of each component depends on fill type, number of layers of reinforcements, modulus of pile and stiffness properties.

2.2 Arching Models

Different soil arching models have been proposed and have been used for design purpose. Different arching model can be summarized as follows;

- Trench model
- Two or three dimensional prism model
- Semi-spherical crown model

Terzaghi (1943) proposed the trench model as presented in Figure 2.3(a). The example for arching of soil by trench model has been illustrated in British Standards BS 8006: 1995 and Miki (1997) have proposed prism model (two or three-dimensional) as shown in Figure 2.3(b). Semi-spherical crown model as proposed by Hewlett and Randolph (1988) is as shown in Figure 2.4(c). Application of single layer geosynthetic in the embankment fill platform is of the common criterion for the above-mentioned soil arching models. In such type of model, a single layer geosynthetics reinforcement acts as a tension member. Collin et al., (2003) illustrates the design procedure for load transform platform for single layer and multilayer geosynthetics in fill platforms. Catenary theory and Beam theory are the two fundamentally different load transfer mechanisms mentioned by Collin et al., (2003). Granular fill is required for multiple geosynthetic layers to form a load transfer platform (Han and Collin, 2005).

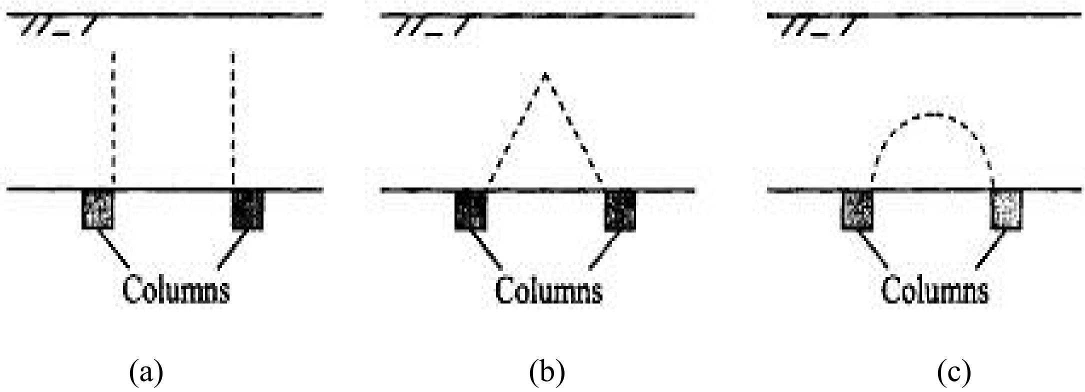


Figure 2.3 Soil Arching Model (a) Trench Model (b) Prism Model
(c) Semi Spherical Model (Han and Collin, 2005)

The response of Arching Ratio as reported by Han and Gabr (2002) is presented in Figure 2.4, Figure 2.5 and Figure 2.6 for fill height, pile modulus and geosynthetic stiffness respectively.

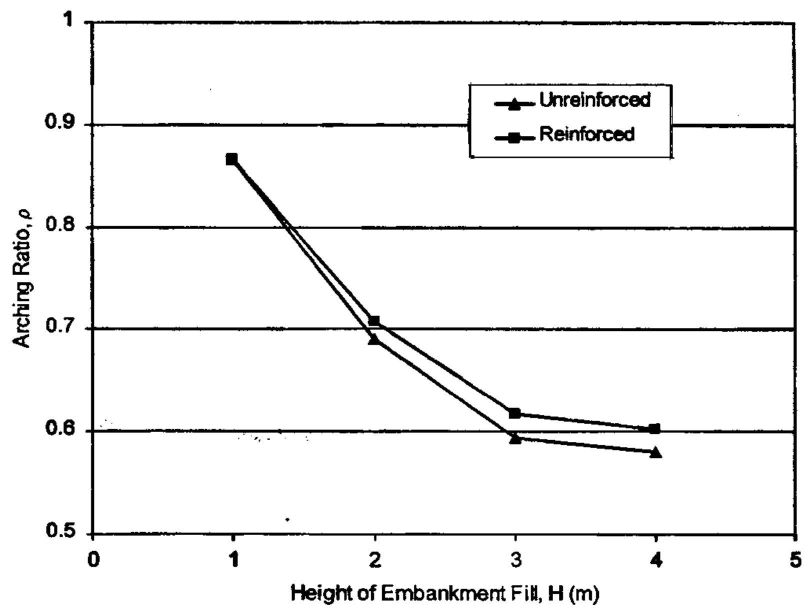


Figure 2.4 Influence of Fill Height on Arching Ratio
(Han and Gabr, 2002)

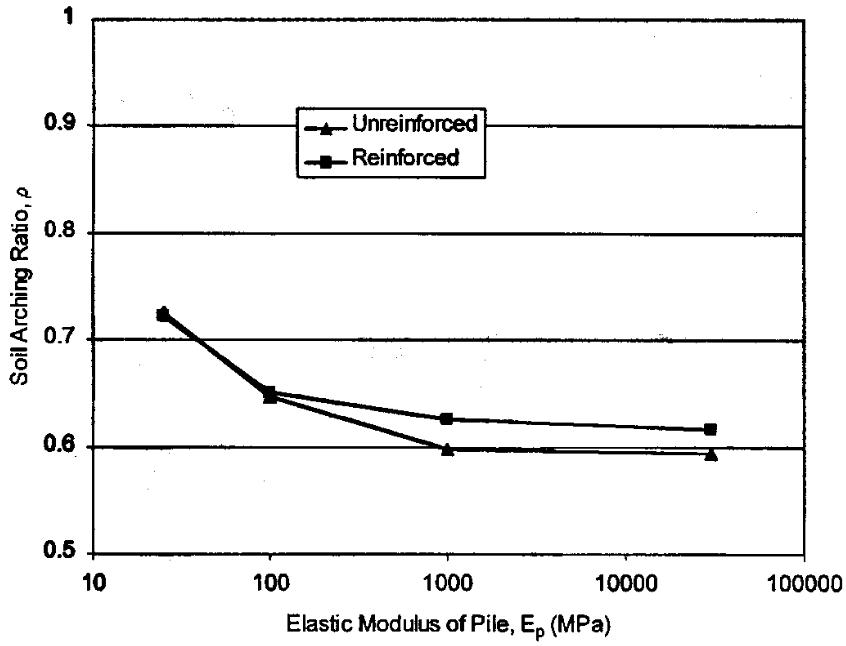


Figure 2.5 Influence of Pile Modulus on Arching Ratio
(Han and Gabr, 2002)

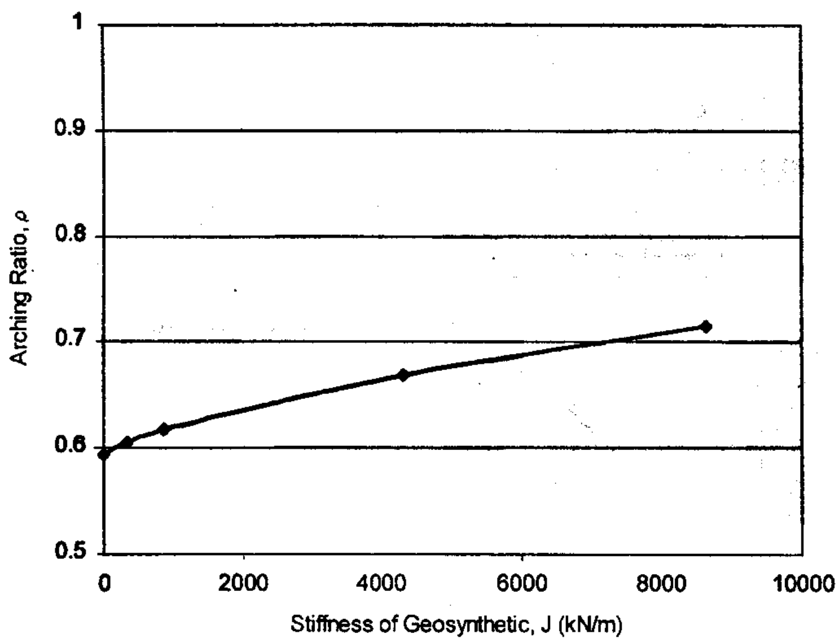


Figure 2.6 Influence of Stiffness on Arching Ratio
(Han and Gabr, 2002)

2.3 Stress Reduction Ratio

Stress Reduction Ratio (SRR), is defined as the ratio of average vertical stress (P_r) carried by the geosynthetic reinforcement to the average vertical stress (unit weight times the fill height) due to the embankment fill. It is assumed that geosynthetics carry the stress applied by the fill embankment. Therefore, SRR value represents the portion of load from the embankment on to geosynthetics.

$$SRR = P_r / (\gamma H) \tag{2.2}$$

Computation of SRR considers factors such as column (pile) diameter, column spacing, embankment fill height, unit weight of fill used, and friction angle of embankment. Five methods have been mentioned in the literature, which can be summarized as;

- British Standard BC8006
- Terzaghi's Theory
- Helwett and Randolph Theory
- Guido Theory
- Carlsson Theory

The above-mentioned methods of computing SRR are based on the assumption that the foundations soil or the soil between the columns (piles) and the geosynthetic layer provides no support. Current design assumes the presence of void below the geosynthetic layer and geosynthetic reinforcement to carry full vertical load from the embankment fill (Miriam and George, 2003). The existing methods to compute the stress reduction ratio are obtainable by square arrangement of columns with square caps as shown in Figure 2.7.

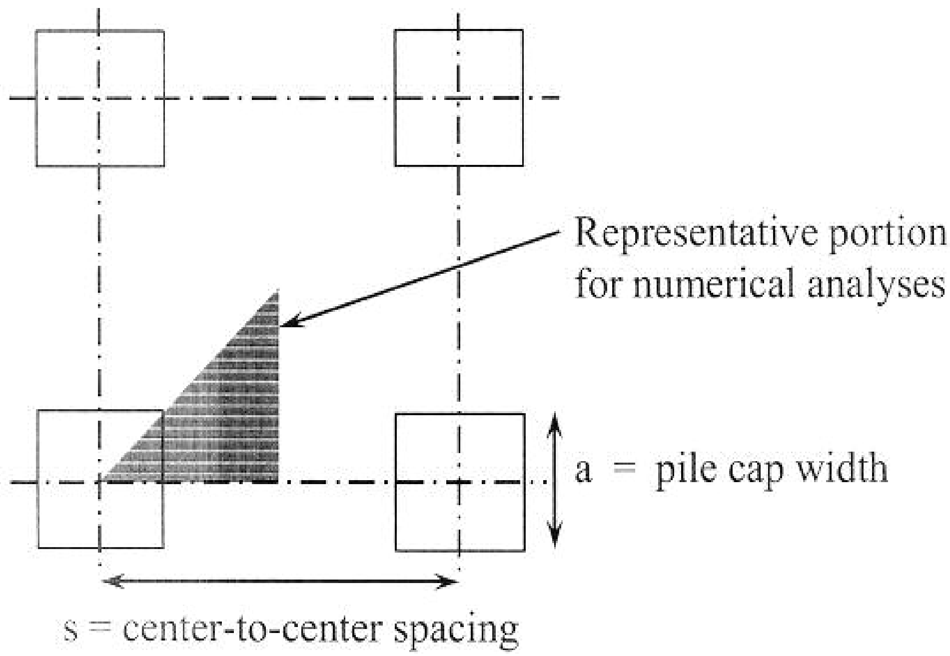


Figure 2.7 Arrangements to Compute SRR
(Miriam and George, 2003)

2.3.1 British Standard BS8006

British Standards BS8006 "Code of practice for strengthened – reinforced soil and other fills – BS8006 1995" implements the design for geosynthetic reinforced pile supported embankment on empirical methods. Jones et al., (1990) developed these design methods. BS8006 considers two methods to evaluate SRR and assumes between the piles. This assumption is to compute the load on geosynthetic. The two methods adopted in computation of SRR according to BS8006 are: (1) When the embankment height is below the critical height of 1.4.(s-a) and (2) The embankment height is more than the critical height. The prior case considered that the arching is not fully developed due to embankment height where as in 2nd case, load above the critical height is assumed to have directly transferred on to the pile system. The expressions for both the cases are as follows:

For H ≤ 1.4(s-a);

$$SRR = \frac{2s(\gamma H + q)(s-a)(s^2 - M)}{(s^2 - a^2)^2 \gamma H} \qquad (2.3)$$

For H > 1.4(s-a);

$$SRR = \frac{16s(s^2 - M)}{(s \ \ + a)^2 H} \qquad (2.4)$$

Where,

$$M = a^2\left(\frac{P_c^{'}}{\gamma H}\right) \qquad (2.5)$$

$$\left(\frac{P_c^{'}}{\gamma H}\right) = \left(\frac{C_c a}{H}\right)^2 \qquad (2.6)$$

C_c = Arching coefficient

= 1.95(H/a) – 0.18 (Non-yielding piles in incompressible stratum)

= 1.70(H/a) – 0.12 (for steel, concrete friction piles and timber pile)

= 1.50(H/a) – 0.07 (for stone, lime columns and sand columns)

2.3.2 Terzaghi's Theory

Russell and Pierpoint (1997) developed a correlation for SRR adapting the arching model developed by Terzaghi. This formulation is developed on the three dimensional nature of column arrangements. The expression for SRR as per the adapted Terzaghi's Theory developed by Russell and Pierpoint (1997);

$$SRR = \frac{(s^2 - a^2)N}{4Hak\tan(\varphi)} \tag{2.7}$$

$$N = 1 - \exp^{\left(\frac{-4Hak\tan(\varphi)}{(s^2 - a^2)}\right)} \tag{2.8}$$

Where;

φ : Friction angle of embankment fill

k : Coefficient of lateral earth pressure

2.3.3 Hewlett and Randolph Theory

Hewlett and Randolph (1988) presented theoretical methods to compute the portion of embankment load, which is applied on to the foundation soils and the columns (piles) through soil arching. This theory is based on limit state of soil in hemispherical domed region over piles as shown in Figure 2.8. The stability of arch crown and at the pile top of the hemispherical domed formed defines the entire stability. The expression for SRR of arch crown and top of pile are given by equation 2.9 and 2.10. The critical SRR or the controlling SRR is the largest between equation 2.9 and 2.10. The fill beneath the dome, between the piles are considered to mobilize no strength which defines isotropic stress state.

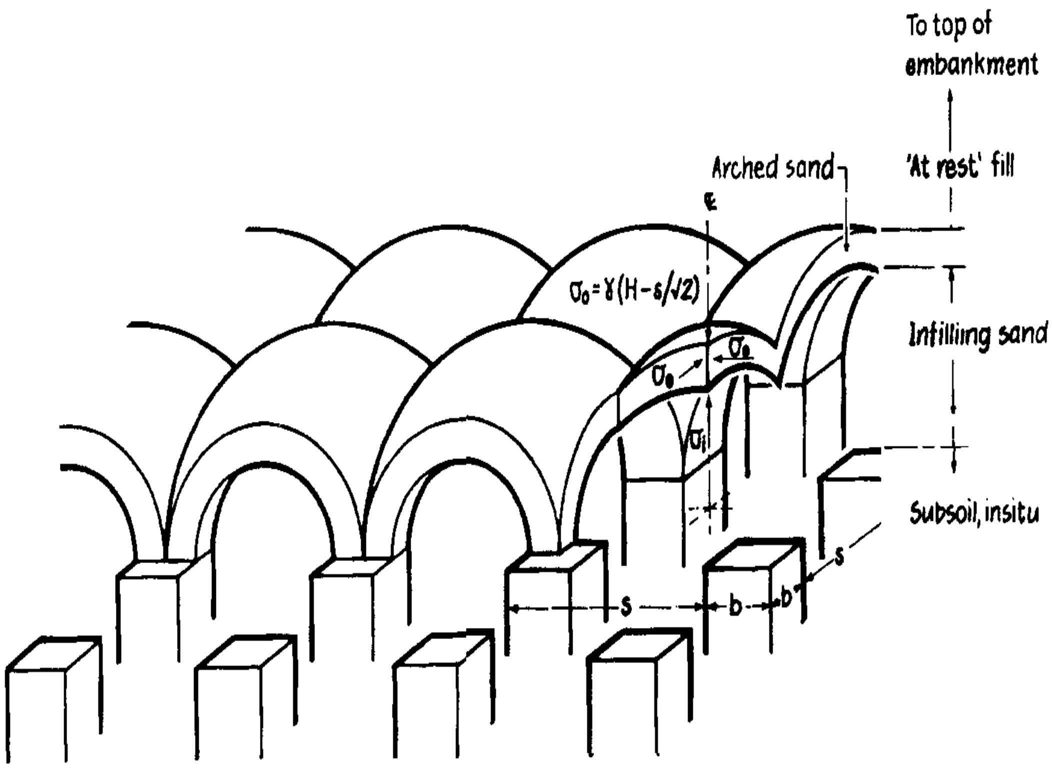


Figure 2.8 Hewlett and Randolph Model (Hewlett and Randolph, 1988)

The expression for coefficient of passive earth resistance (K_p) in accordance with Figure 2.9 is given by the ratio of tangential stress (σ_φ) to radial stress (σ_r).

$$SRR = \left(1 - \frac{a}{s}\right)^{2(k_p - 1)} \left(1 - \frac{s.2.(K_p - 1)}{\sqrt{2}.H.(2K_p - 3)}\right) + \left(\frac{(s-a).2.(K_p - 1)}{\sqrt{2}.H.(2K_p - 3)}\right) \quad (2.9)$$

$$SRR = \frac{1}{\left(\frac{2K_p}{K_p + 1}\right)\left[\left(1 - \frac{a}{s}\right)^{(1-K_p)} - \left(1 - \frac{a}{s}\right).\left(1 + \frac{a}{s}.K_p\right)\right] + \left(1 - \frac{a^2}{s^2}\right)} \quad (2.10)$$

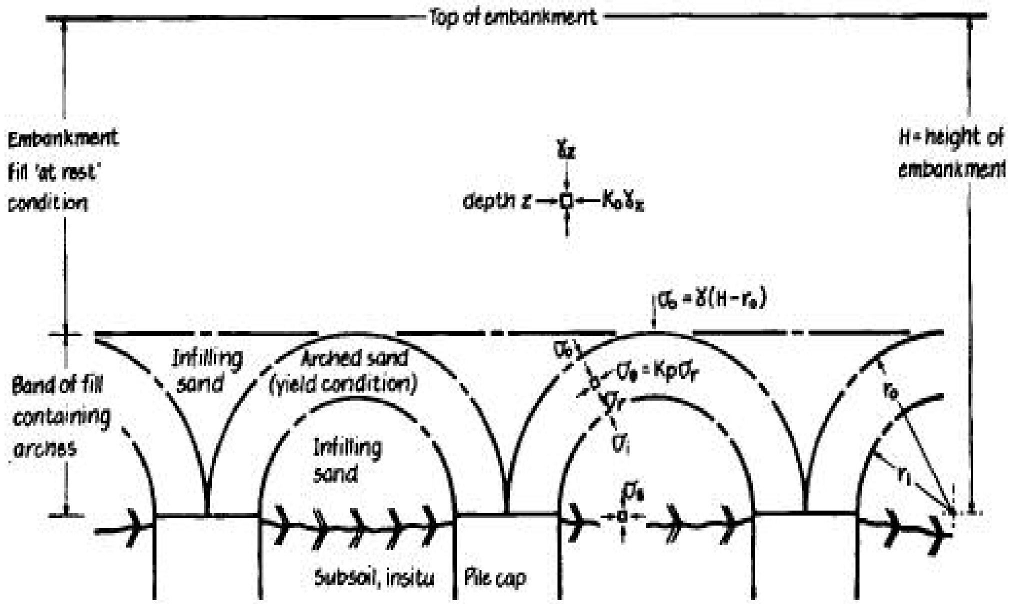


Figure 2.9 Radial Equilibrium (Hewlett and Randolph, 1988)

Where; $$K_p = \frac{(1+\sin\phi)}{(1-\sin\phi)} \tag{2.11}$$

φ : friction angel of the fill embankment

$$\sigma_s = \sigma_i + \gamma(s-b)/\sqrt{2} \tag{2.12}$$

$$\sigma_i = \gamma(H-0.5s)\left[\frac{s-b}{s}\right](K_p - 1) \tag{2.13}$$

$$\sigma_o = \gamma(H - \frac{s}{\sqrt{2}}) \tag{2.14}$$

The pressure on the area between the piles increases linearly as the embankment height increases and no critical height has been observed beyond which the this pressure remains constant as reported by Jerry and George 2003 (Miriam and George, 2003).

2.3.4 Guido Theory

Guido et al., (1987) documented that inclusion of stiff biaxial geogrid within a granular fill will improve the bearing capacity of the foundation soil. Bell et al., (1994) and Guido (1987) concluded that the angle of load spread through a granular fill reinforced with geogrid would be at an angle of 45 degrees. Russell and Pierpoint (1997) adapted these theories to establish expression for SRR. The approach is mainly for a single layer of geosynthetic at the base of the embankment fill. Russell and Pierpoint (1997) presumed the geosynthetic reinforcement carries a pyramid of soil that is not supported by the columns or piles.

The angle of inclination of the edges of the pyramid is assumed to have an inclination angle of 45 degrees. The SRR expression for adapted Guido theory is presented in Eqn. 2.15 (Hewlett and Randolph, 1988).

$$SRR = \frac{(s-a)}{3\sqrt{2}H} \tag{2.15}$$

2.3.5 Carlsson Theory

Carlsson (1987) considered a wedge of soil whose cross-sectional area under the arching soil can be approximated by wedge with an internal angle at the apex of the wedge equal 30 degree. This theory is valid in two-dimensional model. Carlsson adopts a critical height of 1.87(s-a). Miriam and George (2003) present the expression for SRR for this model as in Eqn 2.16 (Hewlett and Randolph, 1988).

$$SRR = \frac{(2s+a)(s-a)}{6(s+a)H\tan(15)} \tag{2.16}$$

Stewart and Filtz (2004), provides a review on magnitude of SRR with varying a/s and H/s ratios as tabulated in Table 2.3 (Hewlett and Randolph, 1988).

Table 2.3 SRR values (Hewlett and Randolph, 1988)

Method	**SRR**					
	a/s =0.25		**a/s = 0.33**		**a/s=0.5**	
	H/s = 1.5	**H/s = 4.0**	**H/s = 1.5**	**H/s = 4.0**	**H/s = 1.5**	**H/s = 4.0**
BS8006	0.92	0.34	0.62	0.23	0.09	0.02
Terzaghi	0.60	0.32	0.50	0.23	0.34	0.13
Hewlett & Randolph	0.52	0.48	0.43	0.31	0.30	0.13
Guido	0.12	0.04	0.10	0.04	0.08	0.03
Carlsson	0.56	0.21	0.48	0.18	0.35	0.13

2.4 Load Transfer Platform

It has been reported that in cases of multilayer (Three layers) geosynthetics reinforcement, maximum tension is developed at the mid span for the lower layer and at the edges of the pile caps for the top layer. The multilayer system of reinforcements acts as a beam for load transfer as shown in Figure 2.10.

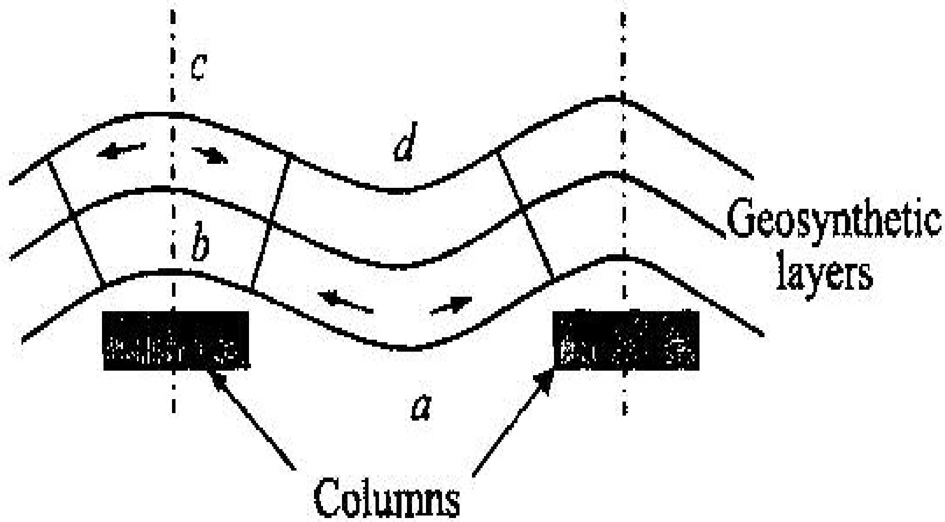


Figure 2.10 Deformation of Multilayer Geosynthetic System
(Han and Collin, 2005)

In beam behavior of a multilayer geosynthetics, interlocking of reinforcements with the surrounding soils increases the stiffness of the composite arrangement. Two main fundamental theories are proposed in designing the load transfer platform. The first approach, followed by British Standard, Swedish and the Germans Standard are documented by Rogbeck et al., (2002). The second approach is mentioned as Collin Method. The prior approach follows “Catenary Theory” whereas Collin Method follows “Beam theory” as shown in Figure 2.11. Collins method is a modified Guido Method as mentioned by Bell et al., 1994 and Hewlett and Randolph 1988. Following are the assumptions in the Catenary Theory (Collin et al., 2003):

- Soil arching forms in the embankment
- Reinforcement deform during the loading process
- Only one layer of reinforcement is modeled

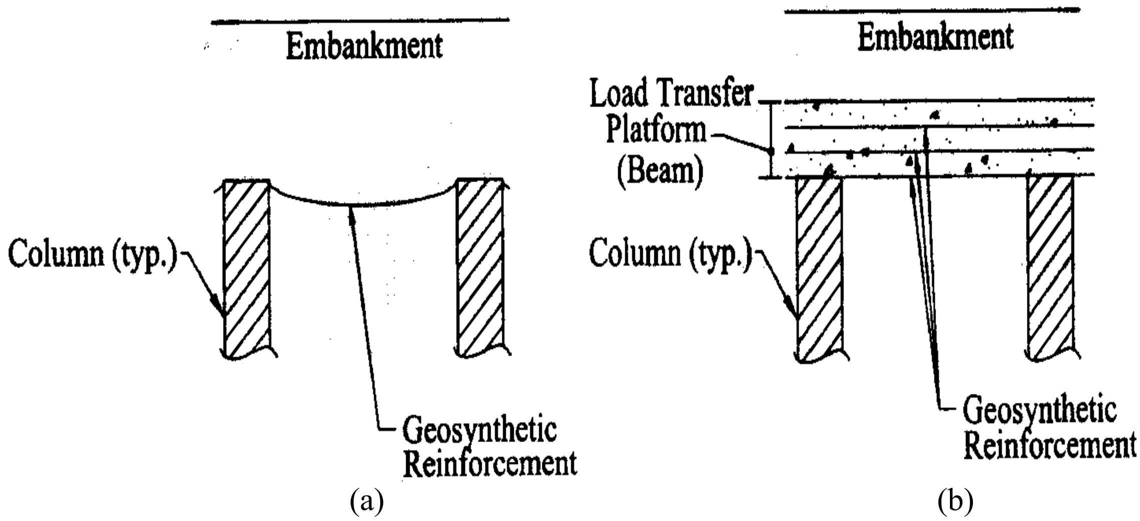


Figure 2.11 Load Transfer Mechanisms (a) Catenary Theory (b) Beam Theory (Collin et al., 2003)

The beam theory is based on some requirements and assumption, which is summarized as follows;

- The thickness of the fill platform is equal to or greater than one-half the clear span between two consecutive columns.
- A minimum of three layers of reinforcement are assumed with a minimum distance of 20cms between two layers.
- The initial strain in reinforcement is limited to 5%.
- The primary function of the reinforcements is to provide lateral confinement of the fill to facilitate soil arching and the secondary function is to support the soil below.

In an idealized case where geosynthetic reinforcement platform is perfectly rigid, no differential settlement is observed which leads to stretching of reinforcement. No embankment soil arching and tension member effect exist in such type of model. It can be established that there is still load transfer or stress concentration from soil to pile due to difference in stiffness in materials.

2.5 Stress Concentration

The degree of load transfer can be quantified by stress concentration ratio (n). Stress concentration ratio is defined as stress on the pile (σ_c) to that of soil (σ_s). Higher the stress concentration ratio, more stress is transferred to the pile. It has been cited that stress concentration ratio increases with height of the embankment. In addition, it confirms that the stress concentration ratio for reinforced section is higher than that of un-reinforced section (Han and Wayne, 2000). The stress concentration ratio increases with increase in tensile stiffness of geosynthetic reinforcement before reaching a constant value. The response of stress concentration ratio as reported by Han and Gabr (2002) is presented in Figure 2.12, Figure 2.13 and Figure 2.14 for fill height, pile modulus and geosynthetic stiffness respectively (Han and Gabr, 2002) .

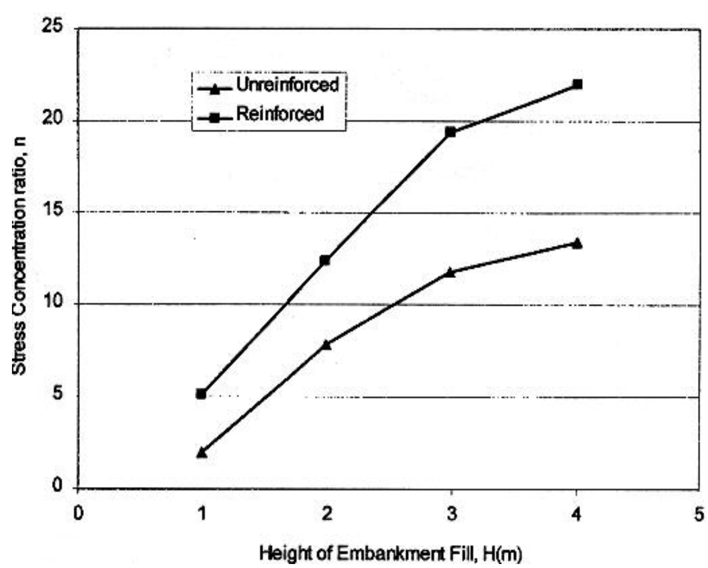


Figure 2.12 Influence of Fill Height on Stress Concentration
(Han and Gabr, 2002)

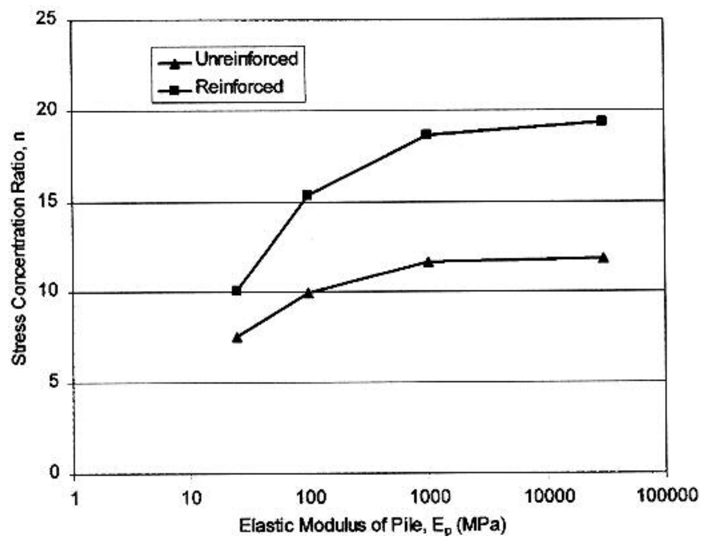


Figure 2.13 Influence of Pile Modulus on Stress Concentration
(Han and Gabr, 2002)

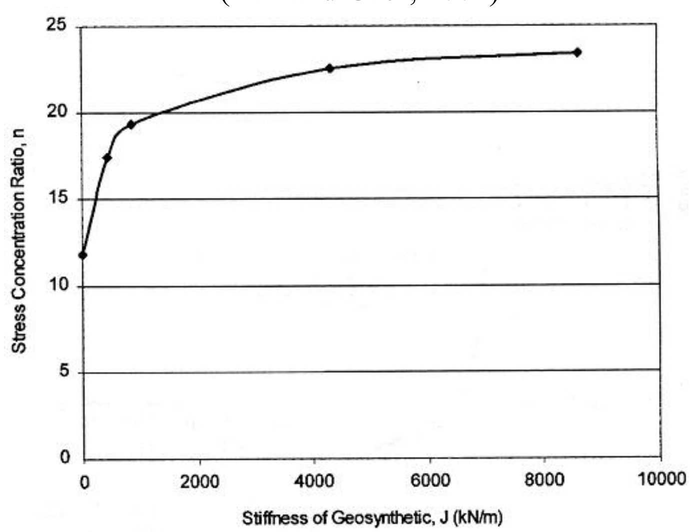


Figure 2.14 Influence of Stiffness on Stress Concentration
(Han and Gabr, 2002)

2.6 Ground Improvement Techniques

A naturally occurring loose and very soft soil requires some engineering treatment to render them fit for bearing the building loads. In absence of any kind of ground improvement, there could be an unacceptable amount of deformation and/or shear failure leading to damage to the structures. To avoid any kind of damage it becomes extremely important to treat or stabilize the soft soil before application of load. Some of the ground improvement techniques include (Geotechnical Design Manual, 2003):

- Excavation and recompaction
- Preloading and surcharge
- Wick drain/sand drain (Dewatering)
- Blast densification
- Dynamic compaction
- Static compaction
- Chemical fixation
- Grout injection/deep mixing technique/soil mix
- Geosynthetics/geosynthetic encased columns
- Lime and cement columns
- Stone/timber columns
- Piling (driven, augured, bored)
- Ground freezing (Temporary improvement)

All of the above-mentioned methods have limitations regarding their applicability and the degree of improvement, which is site specific. In some cases, implementation of two or more techniques in combination may be a suitable solution for very soft soils.

Further discussion on ground improvement techniques are is restricted to deep stabilization technique such as Lime-columns, Deep soil mix, Stone columns, Piling etc.

2.6.1 Soil Mixing and Lime Column Technique

The application of soil mixing for providing stabilization of soft ground is considered as a fast emerging technology in the United States. Settlement control of soft soils under loads can be controlled with treatment ratio of 20% to 30%. As documented in soil mixing reports, Hawaii soft soils were stabilized at 23% treatment ratio as compared to 12% treatment ratio for Florida soils. Soil mix gives satisfactory performance under both static and dynamic loads. As observed and documented by Miki and Mitsuo Nozu (2003) a significant reduction in lateral deformation and settlement due to application of low improvement ratio (Low Improvement ratio Deep Mix- LiDM of about 10-20%) deep mixing method in addition to cost saving is achieved.

Chemico-Pile or chemico-lime (as registered trade name; Onoda Cement Co. Ltd, Japan) column is one of the technique used to improve soft ground. Chemico-Lime is chemically activated quick lime. The constituents of Chemico-Lime are pulverized and granulated quick lime with some additives like calcium silicates or calcium aluminates to accelerate pozzolanic reaction of the composite. The critical effects of application of chemico-lime pile on mixing with soft soils are (Kitsugi and Azakami, 1982);

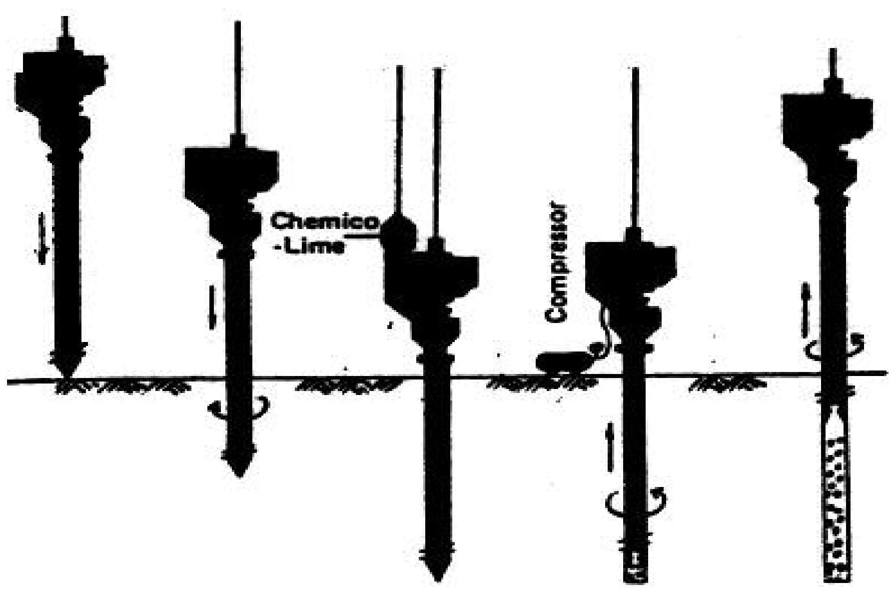
• Rapid Hydration – Quick lime and water combines to form slaked lime. Evaporations of water take place due to generation of heat during the hydration process.

• Volume expansion on slacking of lime which is almost double the volume of quicklime.

• Improvement on Plasticity of soft soils

• Pozzolanic reactions

The above-mentioned process and the cementing properties of materials results in solidification of soil, which in turn increase the load bearing capacity of the soft soils. Typical stage sequence and arrangement of equipment for installation of chemico-lime column is presented in Figure 2.15 and Figure 2.16 respectively.



(a) (b) (c) (d) (e)

Figure 2.15 Construction Stages of Lime Column (a) Casing Placement (b) Drilling (c) Chemico Lime Installation (d) Casing Reversal (e) Lime Column (Kitsugi and Azakami, 1982)

The guide leader of the screw-equipped casing (as shown in Figure 2.16) is installed vertical at the required location on the ground as shown in Figure 2.15(a). By screwing the casing to the desired depth, chemico-lime is introduced into the casing through the table hopper opening as shown in Figure 2.15(b) and Figure 2.15(c). Reversing the direction of rotation of casing and simultaneously increasing the air-pressure in the casing, (with aid of a compressor) desired chemico-lime column is obtained.

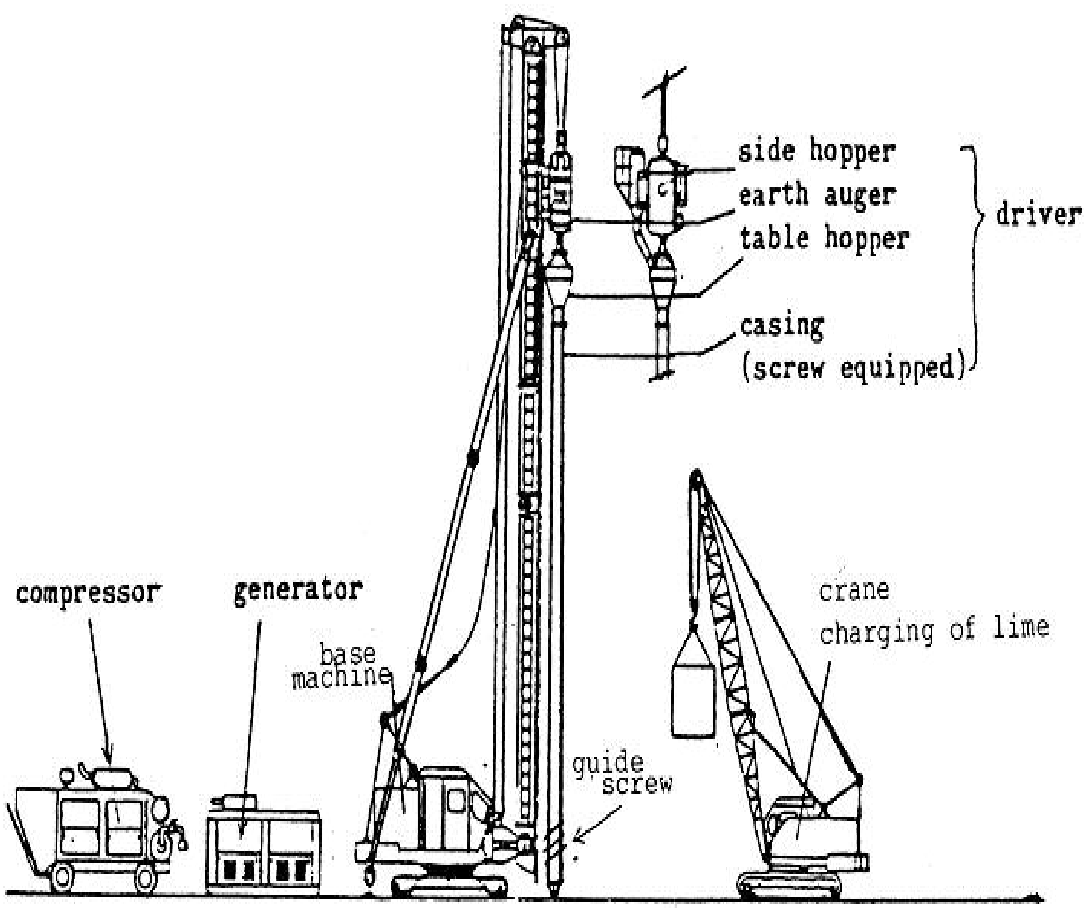


Figure 2.16 Construction Stages of Lime Column (Kitsugi and Azakami, 1982)

2.6.2 Stone Columns

Stone columns are extensively used to improve the bearing capacity of soft soils. The construction of stone column is carried out either by Replacement Method or by Displacement Method (also known as wet method and dry method). The applicability of a particular method depends on the ground water table. If the ground water level is high, in-situ soils being very soft, Replacement Method would be an ideal option. In contrary for low water level and relatively firm soil, Displacement Method is used (Lee and Pande, 1996). The drag forces on the stone columns are mobilized almost immediately and due to favorable conditions for water to drain, consolidation takes places at a rapid phase. The ultimate deformations could be reduced by almost 40% for untreated ground. It is observed that there are no collapses of stone columns. The mode of stone column and relative theories of failure are well documented by Greenwood (1970), Madhav and Vitkar (1978). A stone column may fail due to (a) Shallow shear failure (b) Bulging – Plastic failure and (c) Shear failure in end bearing or skin friction. In case of overload, columns automatically relieve the stress as it deforms. Stone columns are observed to redistribute the load where the stress concentration occurs. A typical stone column tends to perform the following function (Datye, 1980);

- Reduce settlement by reinforcement soil
- Mobilizing the drag forces during initial stage
- Accelerating consolidation process

The two important properties of stone column fills are the maximum angle of friction and measure of stiffness of stone fragments after placement and compaction in the cavity (Poorooshasb and Meyerhof, 1996). Column spacing (area ratio) and degree of compaction govern the efficiency and the performance in terms of strength and stiffness of stone columns. Typical stage sequence for stone column is presented in Figure 2.17.

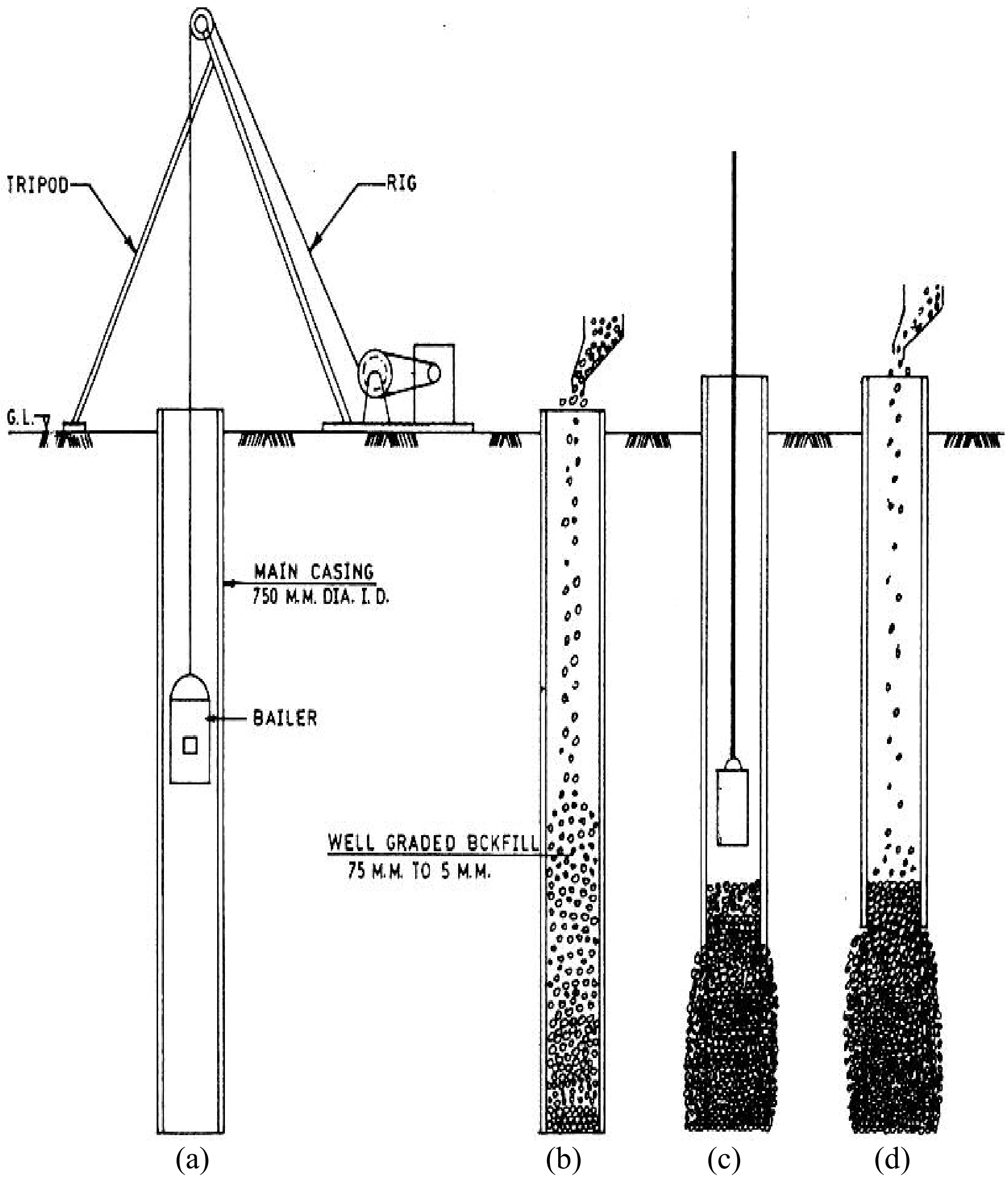


Figure 2.17 Construction Stage of Stone Column
(a) Boring (b) Backfill (c) Casing Withdrawal (d) Stone Column (Nayak, 1982)

In stage (a), boring of Bailer with casing to the complete depth is done followed by filling the cavity with granular fill to a thickness of 2 to 3m as indicated in stage (b). In stage (c), withdrawal of casing is done along with ramming the fill keeping the casing at least 0.3cms below the top of the hammer. This process is continued until the desired length is required.

2.6.3 Piling Methods

The literature on deep foundation methods reveals that the available piles for deep foundations can be classified in different ways as documented by Prakash and Hari (1990). All the methods mentioned in the references can be grouped into following categories ;

- Pile materials
- Method of pile fabrication
- Amount of ground disturbance
- Method of pile installation
- Method of load transfer

The classification based on pile material identifies the pile based on the principal materials, which constitutes a particular pile. Ideal illustrations would be Timber pile, Concrete pile or a Steel pile. The classification based on pile fabrication identifies if the pile is a pre-cast or a cast in-place. Timber and steel piles are always pre-fabricated. However, concrete pile can be pre-cast, pre-fabricated, or cast in-place. Depending on the magnitude of ground disturbance during the piling process, piles can be classified as (a) Large-Displacement piles (b) Small-Displacement piles (c) Non-

Displacement piles (d) Composite piles. Pile based on installation in the ground can be classified as (a) Driven piles (b) Bored Pile (or drilled pile) and (c) combination of Driven and Bored piles. The classification based on the load transfer from the structural member (pile) to the soil consists of (a) End-bearing piles, (b) Friction piles and (c) Combination of end-bearing and friction piles.

Cast-in-place piles are typically installed by in a cavity created either by driving, boring, jetting or coring. Theses piles have many advantages over a precast pile such as (a) Designed for the service loads since they are not subjected to driving or uplift stresses (b) Pile length can be adjusted to suit field requirements (predetermination of pile length is not critical and (c) No additional storage required. Composite concrete piles are composed of combination of steel and timber or timber and concrete or steel and concrete.

2.7 Numerical Modeling

Numerical modeling by finite element or by finite difference is becoming popular. Numerical or statistical model cannot predict the actual in-situ behavior of soil under loading conditions. It is the accuracy of the mathematical tool, which plays a significant role in predicting the behavior of the model. Some of the available numerical analysis and design software are summarized in tabulated 2.4.

Table 2.4 Geotechnical and Geo-Environmental Software’s (Tim, 1996)

Software	Platform	Description
ADINA	WinNT, UNIX, Solaris	2D and 3D static and dynamic, linear and non-linear analysis
BEASY	UNIX, IBM, Sun	2D and 3D analysis, for infinite boundaries, mesh concentration
CRISP-90	DOS	2D and 3D plan strain and axisymmetric, critical state soil model, elastic-perfectly plastic model
DIANA	UNIX, IBM, Sun	2D and 3D plan strain and axisymmetric, critical state soil model, elastic-perfectly
FE2D	DOS	2D and 3D plan strain, plan stress and axisymmetric
FEECON	DOS	2D analysis, plan strain and plan strain using 5 non-linear stress/strain models
FLAC 2D	DOS,UNIX,SUN	2D FD-analysis, 5 non-linear stress/strain models, sequential construction
FLAC 3D	DOS, Windows	3D FD-analysis, simulates large strain behavior, sequential construction

Table 2.4 - *Continued*

IMAGINE	Win95,WinNT	Successor of RHEO-STAUB, code under development
LUSAS	DOS,UNIX	FE code for structural, soil and groundwater modeling
PFC 2D	DOS,UNIX	Partial Flow Code using Distinct Element Method
PLAXIS	DOS,WinNT,Win2000	FEM for plan strain and axisymmetric, 200x15 nodes or 800x6 nodes elements, automatic load stepping, stage construction, FOS analysis
RHEO-STAUB	DOS	2D FEM code, linear arbitrary material model
SAFE	DOS	2D plan strain, plan stress and axisymmetric, change material type while loading
Sage CRISP	Windows	1D\2D and 3D elements, change material type while loading
Sigma-2D	Windows	2D for non-linear, elastic-plastic, contact-surface, plane of weakness, heat-stress and seismic-inertia analysis

Table 2.4 - *Continued*

Sigma-3D	Windows	3D for non-linear, elastic-plastic, contact-surface, plane of weakness, heat-stress and seismic-inertia analysis
SIGMA/W	Windows	FE analysis for stress and deformation analysis
TEMP/W	Windows	FE code for geothermal analysis of problems varying from steady state thermal conduction to transient freeze-thaw
Visage	DOS,UNIX	FE code for geotechnical and rock mechanics, static and dynamic analysis, eigenvalue analysis, seepage analysis
WANFE	UNIX	FE code for non-linear multi-dimensional problems
ZSOIL	Windows,Win95,WinNT	FE method complex geotechnical problems

The finite element analysis methods have been used to analyze the reinforced embankment as found in the literature. Andrawes et al., (1980), Rowe (1982), Boutrop and Holtz (1983), Rowe and Soderman (1984), Rowe et al., (1984), Monnet et al., (1986), Humphery and Holtz (1989), Hird and Kwok (1989) and other have well documented the analysis. Russell and Pierpoint (1997) and Kempton et al., (1998) documented the analysis of embankment with Fast LaGrangian Analysis of Continua modeling program (FLAC – 3-dimensional finite difference program). The output of

FLAC analysis was compared with the design methods previously described by Terzaghi's arching theory. The results were reasonably in agreement with the different arching theory as documented. It is documented that BS 8006 (1995) produces higher SRR value than the FLAC 3-D model for embankment. It was concluded by Russell and Pierpoint that, the reliable design method is by 3-D numerical analysis based on the assumption that soft soil carries no load (Russell and Pierpoint, 1997).

Han and Gabr (2002) analyzed by contrasting several factors like embankment height, tensile stiffness of geosynthetics and pile modulus to correlate the arching ratio and the stress concentration ratio using FLAC 2D. The author concluded that the increase in load transfer resulted due to the increase in embankment height, pile stiffness and the geosynthetic stiffness. It was observed that less stress was applied onto the soft soil due to greater support area considering a 2-D model. Boutrup and Holtz observed the reduction in shear stresses in the soft foundation soil and the reduction in vertical differential settlement at the top of the embankment based on finite element analysis. Maximum shear stress and differential settlement was much more significant when high modulus geosynthetics were used.

Bergado et al., (1994) concluded that both the in-situ measurement data and the finite element analysis by PLAXIS show that the high strength geotextiles can significantly reduce plastic deformations in the foundation soil in addition to the increase in collapse height of embankment due to application of high strength geosynthetics. An increase in the collapse height of embankment was noticed to be 1.5 times the unreinforced case. Bolton seed and John Lysmer studied the impact on finite element analysis on dynamic behavior of soils and soil structures. The analysis was made for different combinations of stress-strain and damping characteristics.

In conclusion, the varying degrees of sophistication of finite element methods have been applied to a wide variety of problems. These applications provide an insight into the nature of role of finite element analysis, which play an important role in non-linear geomechanics designs.

CHAPTER 3

NUMERICAL MODELING

Constitutive modeling of a plastic or elastic perfectly plastic type has been used with success in numerical solution of many complex geomechanics problems. In recent, complete soil behavior under different transient conditions is designed in one single model. Application of sophisticated numerical modeling methods not only improves the reliability on engineering design and provides an economic design.

3.1 Methods of Analysis

The common methods of analysis (solution of differential equations) available to solve complex field problems are presented in Figure 3.1. The recent numerical modeling technique in geomechnaics encompasses analytical and experimental methods, such as laboratory testing and centrifuge modeling. Improvement in computer performance and accessibility has accelerated the lead to development of numerical modeling and simulation to solve complex problems.

Richardson (1910) first introduced the finite-difference method in 1910 on mathematical lines, which was further developed, by Liebman in 1918 and Southwell in 1946 (Frank, 1985). In addition to theories proposed on finite-difference varga amended theory for variational finite differences in 1962, which lead to development of present day finite-element method. In recent times, the technical and economical advantages of numerical modeling are acknowledged both in academics and industry.

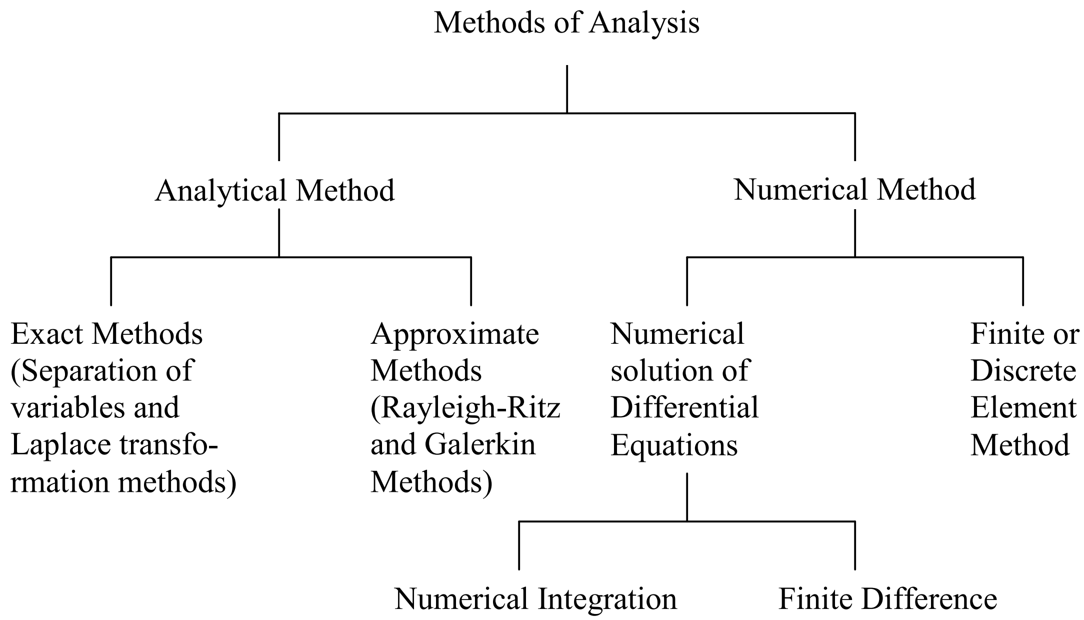
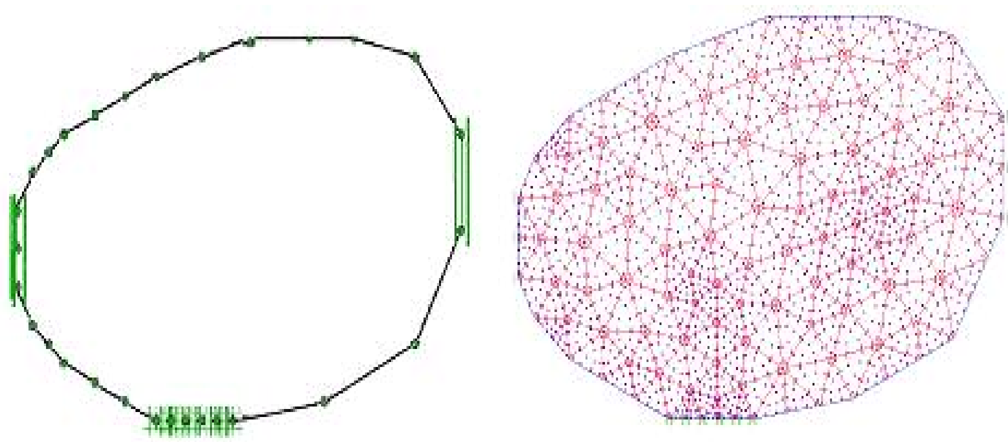


Figure 3.1 Methods of Analysis (Rao, 2005)

The concept is to find an exact solution (and sometimes, even an approximate solution) for a complicated problem by replacing it by a simple problem. The basic idea behind any finite element method is to divide the region, body, or structure being analyzed into a large number of finite interconnected elements. To illustrate, consider a plate (Figure 3.2a) with variable thickness, irregular shape with an uneven loading, to be analyzed for stresses. The field variable in this case is considered as displacement and or deflection and slope. Because of problem simplification (Figure 3.2b), there is infinite number of points in the plate, which results in infinite number of stress to be determined. In such cases, the problem has infinite degrees of freedom.



(a) (b)

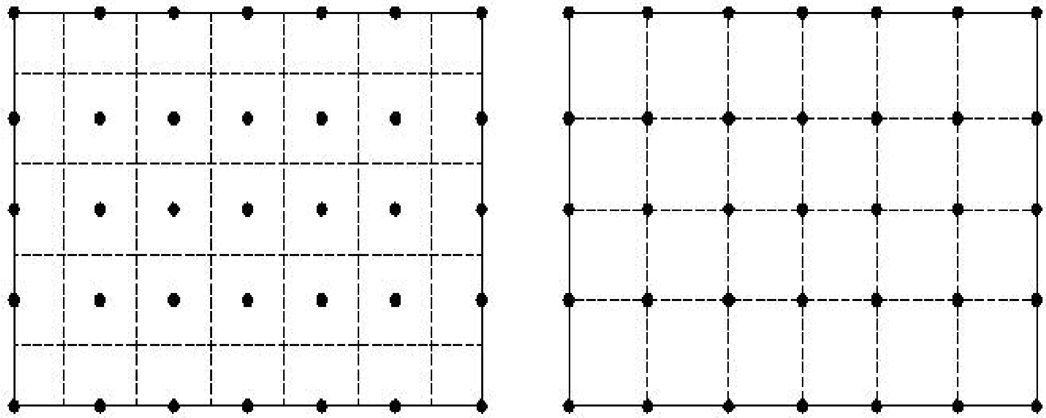
Figure 3.2 Finite Element Mechanism (a) Plate with Variable Thickness, Shape, and Loading (b) Stress Points with Two Dimensional Three-Node Triangular Elements on the Plate

If a closed-form analytical solution exits, in theory we can compute stresses at every point on the plate. An equation in closed-form analytical solution if it solves the problem in terms of functions and mathematical operations from a given accepted set (regular shape is a criterion). Due to irregular shape, computation of infinite stresses precludes the closed-form analytical solutions, which sought for alternative approach. Changing the unreasonable computation of infinite stresses to finite number of points for the analysis (discretization of region) with the aid of interpolation would be a suitable alternative to overcome the deficiency of closed-form analytical solutions.

The key idea of finite element analysis is to (1) discretize complex region into finite elements and (2) use of interpolating polynomials to describe the field variable with in an element (Frank, 1985). Zienkiewicz and Cheung were the first to demonstrate the application of the finite element methods to non-structural problems in the field of conduction heat transfer; however, it was immediately recognized that the procedure was applicable to all problems that could be stated under variable form. These concurrent developments made the finite element analysis as one of the most powerful approximate solution methods in recent times (Frank, 1985).

3.2 Fundamentals of Discretization

The finite-difference method (FDM) is a best alternative solution technique, which can be used to solve the same complicity problems as in the case of finite element analysis (FEM). Both the methods require discretization of complex problems however; the discretization method is done is fundamentally different. Restricting the discussion to the two dimensional rectangular elements, in FEM analysis the nodes are considered at the corner of the rectangular element. However, in FDM analysis the discretization is done by dividing the region to be analyzed into finite number of lumps (Frank, 1985). In the FDM approach, each lump is considered to have constant value of field variable in contrast with the FEM analysis where a rectangular two-dimensional element is analyzed with four nodes at the corner of the rectangle having different values of field variables. In general, the nodes (corners in cases of FEM and center in case of FDM for a rectangular two-dimensional element) are the locations at which field variables are to be determined. Figure 3.3 illustrate the concept of discretization in FDM and FEM.



(a) (b)

Figure 3.3 Discretization of Rectangular Plate (a) Lumps for Finite Difference Analysis (b) Nodal Points for Finite-Element Analysis (Frank, 1985)

3.2.1 Convergence

Eudoxus and Cnidus were the first to determine the value of л by the method of exhaustion. Exhaustion is a process in which the area bound by one or more curves is computed by replacing it with a simple one and further integrating the individual values in the required area. To illustrate the method of exhaustion consider a curve as shown in Figure 3.4. Area enclosed by the curve is obtained by dividing the area by number of rectangles and summing the individual values (Figure 3.4b). The bound concept originated from this process as illustrated in Figure 3.4b and Figure 3.4c. The inscribed rectangle would give a lower bound where as the circumscribed rectangle would give an upper bound. As more and more rectangles are considered in the analysis, the better would be the approximate value, which provides a platform for the convergence theory (Frank, 1985).

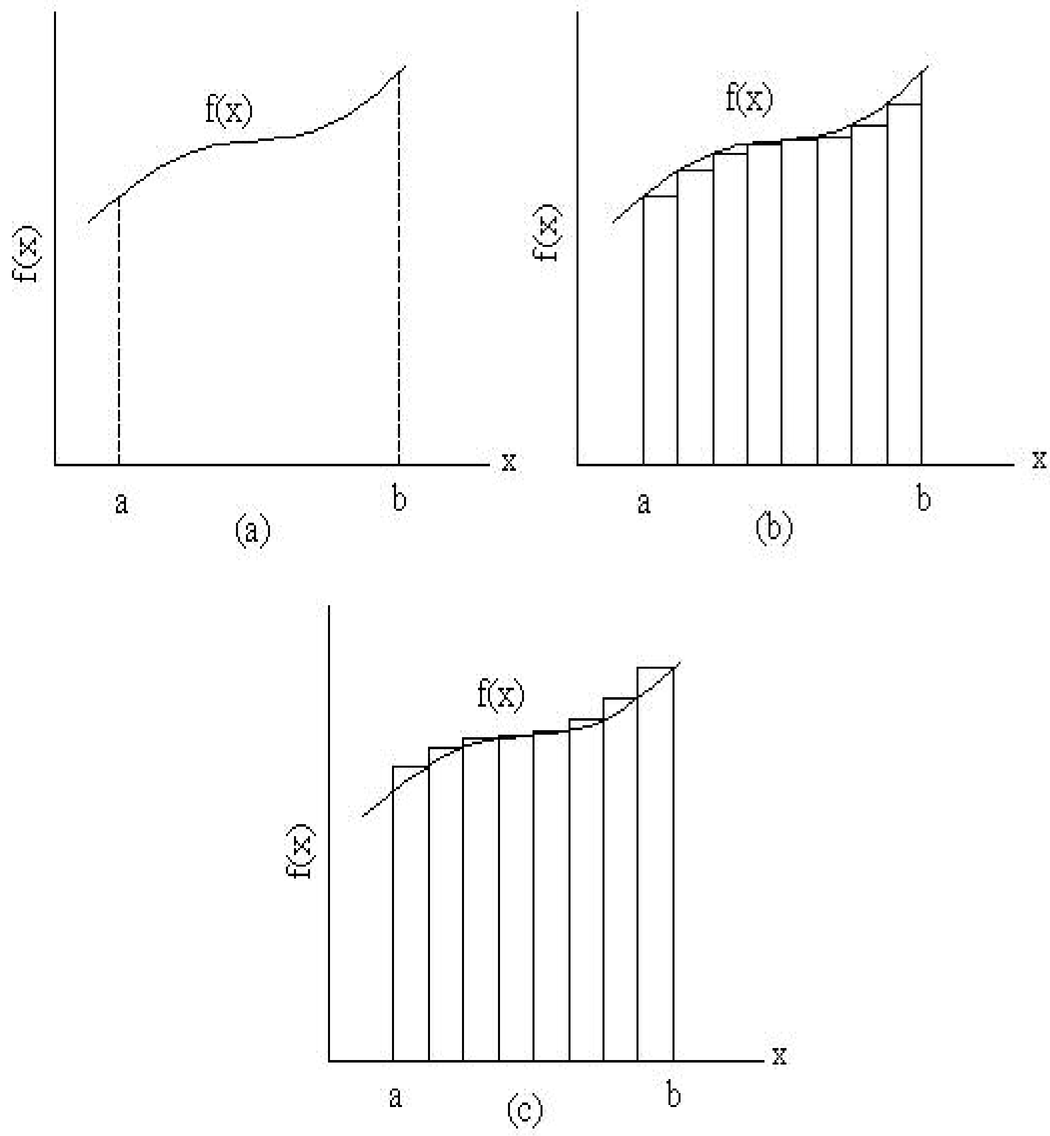


Figure 3.4 Method of Exhaustion to Find Area Under Curve (a) Exact Area (b) Lower Bound by using Inscribed Rectangles (c) Upper Bound by Circumscribed Rectangles (Frank, 1985)

3.2.2 Isoparametric Elements

Finite-difference nodal points are written with basic laws of thermodynamics (conservation of energy) or Newton's second law, which is relatively straightforward.

However finite element method adopts direct energy balance approach in addition to principle of virtual work, variational methods, and weighted-residual method. Weighted-residual method and principle of virtual work is extensively used for non-structural and structural applications respectively. The accuracy of convergence theory or the accuracy of the approximate solution method is illustrated in Figure 3.4 Also, illustrating the use of higher order element such as rectangular and quadrilateral along with the classical two-dimensional triangular elements.

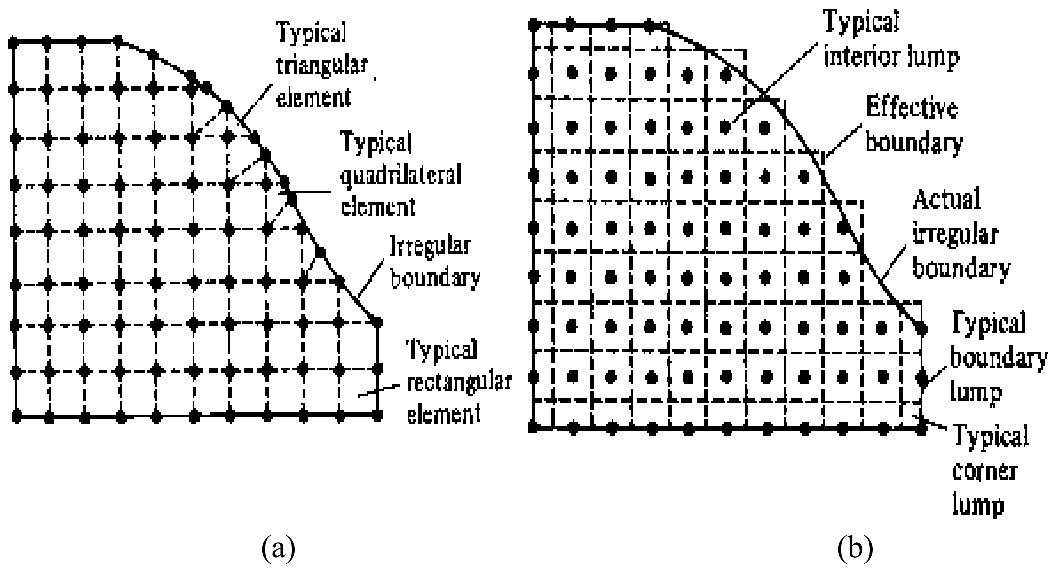


Figure 3.5 Irregular Shape Plate (a) Use of Higher Order Element (b) Lumps (Frank, 1985)

The use of higher-order element, which requires higher-order interpolating polynomials in a FEM, proves that the FEM is accurate when curved boundaries need to be analyzed. Some of the higher-elements used in the finite element method are illustrated in Figure 3.5 and Figure 3.6.

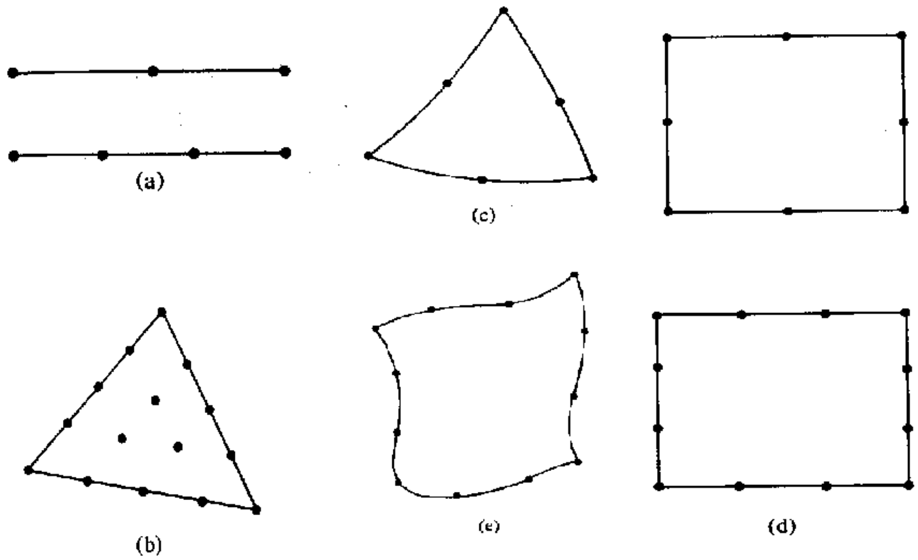


Figure 3.6 Higher Order Elements (a) One-Dimensional (b) Two-Dimensional (c) Two-Dimensional with Curved Sides (d) Two-Dimensional with Rectangular Sides (e) Two-Dimensional Quadrilateral with Curved Sides
(Frank, 1985)

3.3 PLAXIS - Finite Element Modeling

3.3.1 PLAXIS General

Development of PLAXIS began in 1987 at the Technical University of Delft. The initial goal was to develop an easy-to-use two-dimensional finite element code for river embankment analysis on soft soils. Because of continuous development, a company named PLAXIS B.V was established in 1993 and the first PLAXIS version for Windows was released in 1998 with a main objective to provide a straightforward but theoretically rigorous tool for all practical analysis in geotechnical engineering (PLAXIS, 2000).

3.3.1.1 Model

Two-dimensional finite element analyses can be performed either with plane strain or axisymmetric model. A plane strain model is used for geometries with uniform cross section and corresponding stress state and loads over certain length perpendicular to the cross section. However, axisymmetric models is used for circular structures with a uniform radial cross section and loads around the central axis. Both the models results in two dimensional finite element model with two translation degrees of freedom along x-axis and y-axis. In the numerical model calibration and model analysis, plane strain analysis is used.

3.3.1.2 Element Type

To model soil layers and other clusters, a 15-node or 6-node triangular elements may be used. A 15-node element provides fourth order interpolation for the variable field (displacements) and the numerical integration involves twelve Gauss points also known as stress points. However, for a 6-node element, order of interpolation is two and numerical integration involves three Gauss points. A 15-node triangular element is preferred over a 6-node element for its very accurate and high quality stress results. In addition, it has been observed that the 6-node element over predicts the failure loads and safety factors. A beam and a geotextile (structural element) are 5-node and 3-node elements, which is compatible with the 15-node or 6-node soil elements. Since the failure due to excessive settlement is a concern in the present model, a 15-node triangular element is considered for the analysis.

3.3.1.3 Interface Model

Interface has a ‘virtual thickness’ with imaginary dimensions used to define the material properties of the interface. In general, an interface is supposed to generate very small elastic deformation, which makes the virtual thickness to be small. In other words, it can be inferred that lesser the virtual thickness lesser is the elastic deformation. In PLAXIS, the stress-strain behavior at soil-structure interface is simulated by elastic-perfectly plastic model. The model parameters at the soil-structure interface can be modeled from the soil with a correlating factor called interaction co-efficient or interface strength, R_i or R_{inner}. The interface properties are computed from the soil properties as follows:

$$c_i = R_i \, c_{soil} \tag{3.1}$$

$$\tan\varphi_i = R_i \tan\varphi_{soil} \leq \tan\varphi_{soil} \tag{3.2}$$

$$\psi_i = 0^0 \text{ for } R_i < 1 \text{ and } \psi_i = \psi_{soil} \tag{3.3}$$

$$G_i = R_i^2 \, G_{soil} \leq G_{soil} \tag{3.4}$$

Where c_i, φ_i, ψ_i and G_i are the cohesion (adhesion), friction angle, dilatancy and shear modulus of the soil-structure interface. Therefore, if the cohesion (c), friction angle (φ_{soil}), dilatancy (ψ_{soil}) and shear modulus (G_{soil}) of soil is known, the corresponding soil-structure interface properties can be generated using the parameter R_i which is usually 2/3 for numerical modeling. In the present model since the shape of the column does not remain straight as in theory, having high lateral deformation the numerical value of interface is considered as one.

3.3.1.4 Mesh

The geometry has to be divided into finite elements in order to perform finite element analyses. Division of geometry is automatically done when all the material properties and the structural elements are defined in the model. A composition of interconnected finite elements is called mesh. Basic type of element in a mesh is the 15-

node triangular element or the 6-node triangular element. A 15-node triangular, un-structured mesh element is considered in the present modeling. The mesh generator require a meshing parameter which represents the average element size l_e. Meshing parameter is dependent on the geometry dimensions of the model and the coarseness factor called n_c. The relation between the average element size, meshing parameter, and the coarseness factor is shown below (PLAXIS, 2000):

$$l_e = \sqrt{\frac{(x_{max} - x_{min})(y_{max} - y_{min})}{n_c}} \tag{3.5}$$

Where $x_{max,}$ x_{min}, y_{max}.and y_{min} are the outer geometry dimensions. and n_c is given by:

Very coarse	n_c =25	Around 50 elements
Coarse	n_c =50	Around 100 elements
Medium	n_c =100	Around 250 elements
Fine	n_c =200	Around 500 elements
Very fine	n_c =400	Around 1000 elements

3.3.2 Modeling Soil Behavior

In PLAXIS, the available soil models are Linear Elastic model (LE), Mohr-Coulomb model (MC), Jointed Rock model (JR), Hardening Soil model (HS), Soft Soil model (SS), Soft Soil Creep model (SSC) and User-Defined model (UD). Among all the above-mentioned models, Mohr-coulomb model is considered as the first order approximation of real soil behavior. This elastic perfectly plastic model requires five basic soil input parameters, namely young’s modulus (E), poisson’s ratio (ν), cohesion (c), friction angle (φ) and dilatancy angle (ψ). In general, all model parameters are meant to simulate the effective soil state. An important soil property would be presence of pore water. Pore pressure significantly influences the soil response. To incorporate the pore pressure effects three types of behaviors have been facilitated in the software: drained behavior, undrained behavior and non-porous behavior. A drained behavior is used when no excess pore pressure is generated in case of dry soils, with full drainage due to high permeability and/or low rate of loading. This option is perfect to simulate long-term soil conditions. An undrained behavior is opted to develop excess pore pressure. To neglect both initial and excess pore pressure in the model, non-porous behavior is used. This application is found in modeling concrete or structure behavior. Two types of analysis is normally adopted in PLAXIS analysis namely undrained analysis with effective stress parameters and undrained analysis with total stress parameters. If the effective stress parameters are known it is possible to specify undrained behavior using effective model. However if accurate effective parameters are not available, it is possible to perform a total stress analysis with φ_u =0, in the Mohr-coulomb material mode. This type of analysis is also known as c_u analysis.

3.3.2.1 Mohr-Coulomb Model

Mohr-Coulomb model, is highly recommended where soil parameters are not known with great certainty. This model requires five basic soil input parameters, namely young's modulus (E) or shear modulus (G), poisson's ratio (ν), cohesion (c), friction angle (φ) and dilatancy angle (ψ).

3.3.2.1.1 Shear Modulus

The shear modulus (G) is defined as the ratio of shearing stress to shearing strain, which is closely linked to the young's modulus (E), and the poisson's ratio (ν). The correlation is as shown below:

$$G = \frac{E}{2(1+\nu)} \qquad (3.6)$$

In soil mechanics, the initial slope is usually indicated as E_0 and the secant modulus at 50% strength and is denoted by E_{50}. It is highly recommended to use E_0 value for over-consolidated clays and some rocks and E_{50} value for sands and near normally consolidated clays. It is a natural tendency for the initial modulus and the secant modulus to increase with an increase in confining pressure i.e deep soils tend to have greater stiffness than shallow layers. It is possible to estimate the young's modulus for cohesive soils considering the sensitivity as shown below (Joseph, 1997)

Normally consolidated sensitive clay:

$$E_s = (200 \text{ to } 500)\ S_u \qquad (3.7)$$

Normally consolidated insensitive and lightly overconsolidated clay:

$$E_s = (750 \text{ to } 1200)\ S_u \qquad (3.8)$$

Heavily overconsolidated clay:

$$E_s = (1500 \text{ to } 2000)\ S_u \qquad (3.9)$$

In general

$$E_s = \alpha\ S_u \qquad (3.10)$$

In case of Nong Ngo Hao clay, the value for proportionality constant in equation-3.10 is considered between 75-100. Hence, in the numerical model the value of α is considered between 75-100.

3.3.2.1.2 Poisson's Ratio

The ratio of axial compression to lateral expansion defines poisson's ratio. Selection of poisson's ratio is simple when the elastic model or Mohr-coulomb model is used for gravity loading.

$$\frac{\sigma_h}{\sigma_v} = \frac{\nu}{1+\nu} \tag{3.11}$$

As both the model provides a well-known ratio for one-dimensional compression it is easy to select an appropriate value which gives a realistic value of Ko. In most of the cases, the value of poisson's ratio is considered between 0.3 to 0.4.

3.3.2.1.3 Cohesion

PLAXIS can handle both cohesionless soils and cohesive soils. In a non-linear analysis it is highly recommended to use a small value cohesion (c between 0.2 kPa - 1kPa). An advanced parameter option is available in the software in which the increase of shear strength or cohesion with depth can be simulated.

3.3.2.1.4 Friction Angle

The friction angle largely determines the shear strength by Mohr's stress circles. It is highly recommended to use the critical state friction angle rather than using higher value obtained from small strain. The computational time increases more or less exponentially with increase in friction angle (PLAXIS, 2000).

3.3.2.1.5 Dilatancy Angle

The dilatancy angle in case of heavily over-consolidated layers, clay soils tends to show no or negligible magnitude. The dilatancy angle for sand depends both on the density and the angle of internal friction. A small negative value of dilatancy angle is realistic for loose sand. A basic equation for dilatancy angle and the friction is shown below:

$$\psi \approx \varphi - 30^0 \tag{3.12}$$

3.3.3 Finite Element Model Calibration

It is necessary to device a mathematical model that is capable of simulating the response to prescribed actions such that acceptable agreement between predicted results and observations of physical can be obtained (Meyer, 1987). The process of standardizing, modifying, and verifying mathematical model can take several forms. A common engineering practice is to construct a mathematical model and predict with its output. On satisfactory agreement of the predicted output with the physical experiments confirms correctness of both mathematical model and the physical test. Hence, model calibration is considered mandatory for numerical analysis.

3.3.3.1 Model Simulation

PLAXIS version 8 has been used for the current analysis of Pile Supported Embankment. The finite element model along with lower boundary as fixed and two sides as roller supports and the finite element mesh for the model is presented in Figure 3.7 and Figure 3.8 respectively. The lower boundary is considered as no-draining side due to presence of hard soil where as the two, vertical sides are taken as the draining sides. The finite element is simulated with 4 layers of soil profile consisting : 0-1.5 m of Chemicolizer, 1.5-8.5 m of Very Soft Clay, 8.5-11.5 m of Soft Clay and 11.5-16 m of Medium Clay. An embankment fill is being considered in two layers as step loading of one meter each.

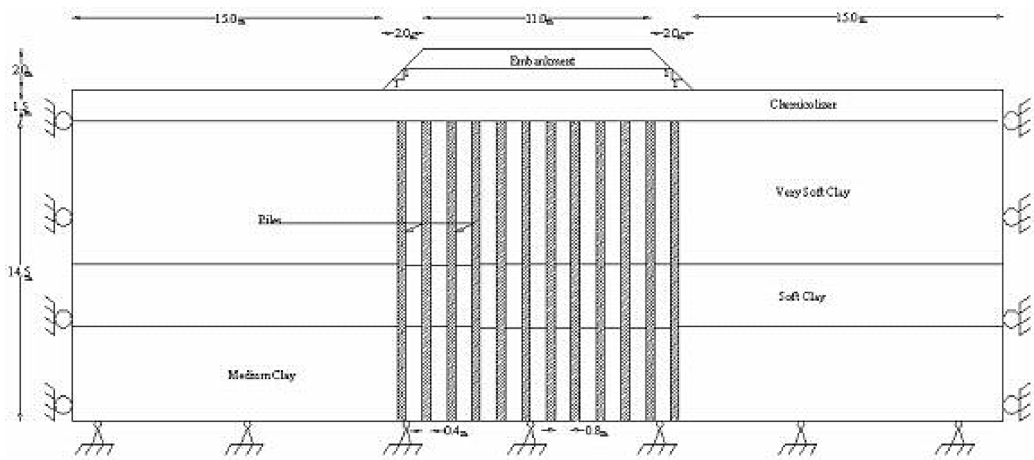


Figure 3.7 Finite Element Model of Pile Supported Embankment (Hossain, 1997)

The lime pile of diameter 0.4m and 16m length have been simulated as 5-node beam element. The water table is taken at the ground surface. The 15-node finite element triangular mesh is generated in accordance to the profile mentioned above as illustrated in figure 3.7.

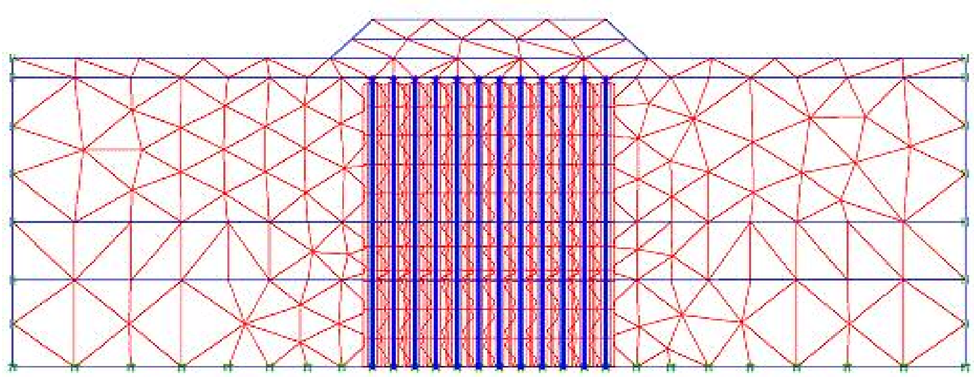


Figure 3.8 Finite Element Mesh of Pile Supported Embankment

3.3.3.2 Material Model and Parameters

3.3.3.2.1 Foundation Soil

The selection of model for the analysis is to done mainly based on availability of soil parameters. In the case of Soft Soil Model (SS), care should be taken with respect to overconsolidation ratio (OCR) and co-efficient of lateral stress (Ko). Poor estimation of the OCR and Ko will significantly affect the stiffness of the soil layer. The Hard Soil Model (HS) and Soft Soil Creep Model (SSC) have limitations for soils that are intermediate between being soft and hard. Mohr-Coulomb model, a first approximation of soil behavior is highly recommended where soil parameters are not known with great certainty. Model parameters for foundation soil in finite element analysis are tabulated in Table 3.1 and Table 3.2. The parameters are taken from data of the Onoda Cement Progress Report (Hossain, 1997).

Table 3.1 Material Properties for FEM Analysis (Hossain, 1997)

Depth of soil	Material Set	Drainage	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_x (m/d)	k_y (m/d)	R_i
0-1.5 m	Chemicolizer	Undrained	15	22	0.009	0.0009	1
1.5- 8.5	Very soft Clay	Undrained	16	16	0.005	0.005	1
8.5-11.5	Soft clay	Undrained	17	17	0.0009	0.0009	1
11.5-16.0	Medium Clay	Undrained	18	18	0.0007	0.0006	1

Table 3.2 Parameters for Foundational Soil (Hossain, 1997)

Depth of soil	Model	ν	G (kPa)	E (kPa)	C(kPa)	φ(degree)
0-1.5 m	M-C	0.3	7692.3	20000	200	0
1.5-8.5	M-C	0.3	807.69	2100	21	0
8.5-11.5	M-C	0.3	884.61	2300	23	0
11.5-16.0	M-C	0.3	1115.38	2900	29	0

3.3.3.2.2 Embankment Fill

An embankment fill is being considered in two layers as a step loading of one meter each. The water table was taken at the ground surface. The elastic-perfectly plastic, Mohr-Coulomb model was used to analyze the model. The properties of the fill materials are summarized in Table 3.3. The parameters are taken from data of the Onoda Cement Progress Report (Hossain, 1997).

Table 3.3 Parameters for Embankment Soil (Hossain, 1997)

Embankment	Model	ν	G (kPa)	E (kPa)	c (kPa)	φ(degree)
2m height	M-C	0.3	3076	8000	1	30

3.3.3.2.3 Pile Properties

The lime pile of diameter 0.4m and 16m length have been simulated as 5-node beam element. In formulating the stress-strain behavior at the soil-structure interface, the thickness was assumed as 0.1 to 0.01 times of the length of corresponding interface element. The corresponding soil-structure interface properties can be generated using the parameter R_i which is usually 2/3 for numerical modeling. In the present model, since the shape of column does not remain straight as in the theory, having high lateral deformation numerical value of interface is considered as one.

3.3.3.3 Loading of the Embankment

Embankment fill is being considered in two layers as the step loading of one meter each according to the construction sequence. The water table is taken at the ground surface. It has been considered that the first loading of 1m of the improved ground did not take any time and it has been applied instantaneously for 2 days. After this period, the second 1m of the load has been applied and this total 2m of load continued for 42days.

CHAPTER 4

NUMERICAL MODELING OF PILE SUPPORTED EMBANKMENT

4.1 Introduction

The stabilization of soft clay is one of the most important construction techniques in geotechnical engineering. Soil stabilization refers to the collective term for any physical, chemical, or biological methods used to improve the engineering properties of a natural soil to make it serve adequately an intended engineering purpose. The design of embankments on weak foundation soils is a challenge to the geotechnical engineer. The injection method using chemical grouting in the ground (chemical injection grout) has been widely applied to improve weak ground. Chemical admixture (lime and cement as chemical admixture) stabilization has been extensively used in both shallow and deep stabilization in order to improve inherent properties of soil behavior such as strength and deformation. This process was developed simultaneously in Sweden and Japan in the 1970's. Cement/Lime columns, using Dry Jet Mixing (DJM) which pneumatically delivers cement or lime powder into ground and mixes it with soil to form soil-cement/soil-lime column, were preliminary executed and reported to be successfully in practice in 1980 and 1983 (DJM research Group (1984); Chida (1982)a & (1982)b; Miura et al., (1986); Cox (1981); Broms (1986)). Today the method is used worldwide, especially in Europe, North America, and Asia.

A summary of mixing methods adopted for ground improvement is presented in Figure 4.1.

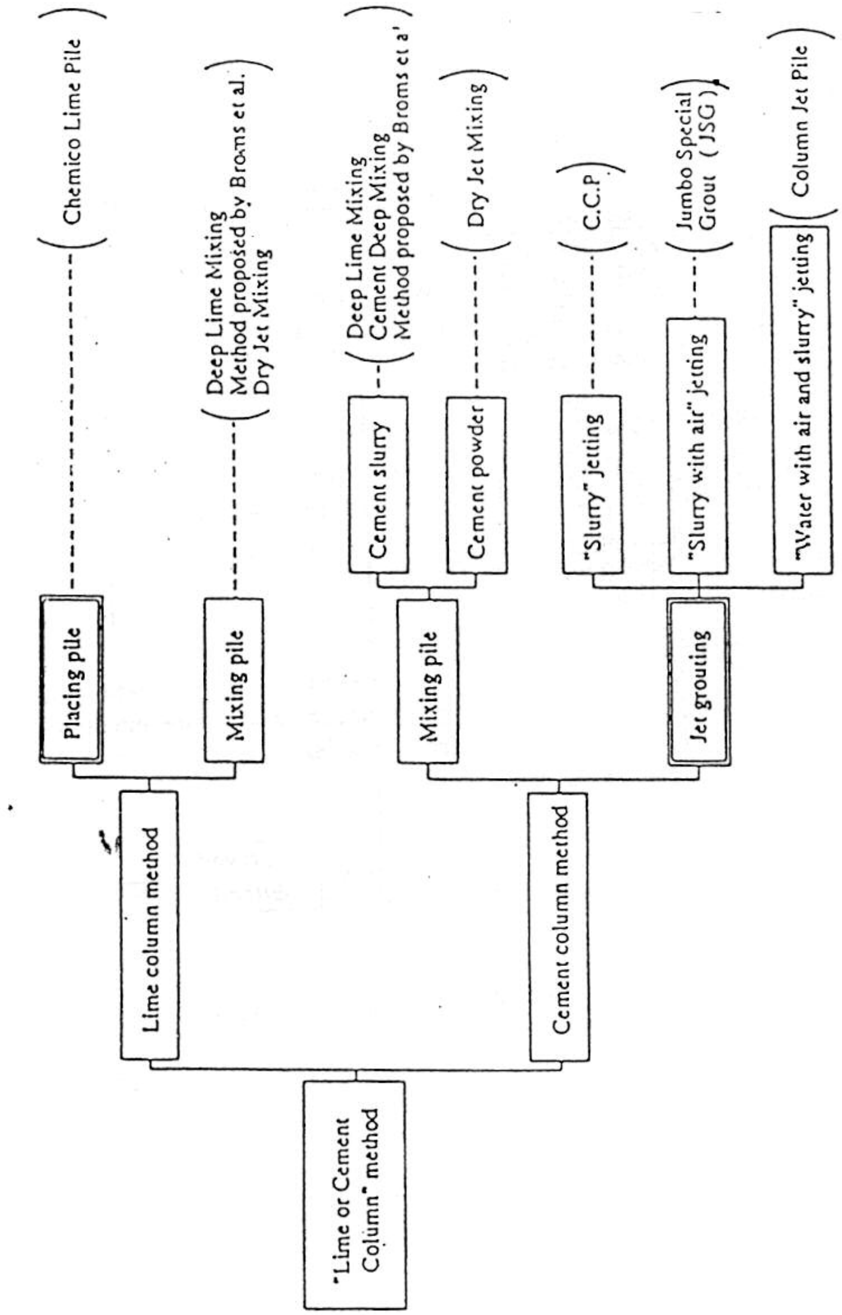


Figure 4.1 Deep Mixing Method (Takeshi et al., 1992)

There are two methods of improving soft clayey ground by using lime; (a) Dry Jet Mixing (DJM), involves mixing lime with the soil to form lime columns and which are widely used in Scandinavian countries (Broms, 1985). (b) Installations of chemically treated lime columns in the soil without mixing. This is the called as "chemico-lime pile" or "chemico-pile" method, which is widely used in Japan, Singapore and Scandinavian countries. The effects of this method on very soft soils are: (i) Rapid reduction of water content of the soil surrounding the chemico-pile, and (ii) Quick increase of lateral compaction in the soil foundation by means of volume expansion of this column.

The chemico-pile method is a fast, effective, and well-recognized technique for ground improvement in soft clay deposit. Anshumali (1981) conducted consolidometer tests to investigate the compressibility effects of the lime column on Bangkok clay. The test results showed significant improvement on soil compressibility behavior. The shear strength of the lime column were increasing twice or thrice with the course of time. Lee (1983) affirms Anshumali (1981) observations for the soils having 4% organic content.

Broms (1984) reported that the final undrained shear strength of the lime columns could be as high as 10 to 50 times the initial shear strength (10 to 15 kPa). The shear strength of stabilized soil increases approximately linearly with time when plotted on a log-log diagram. Brandl (1981) observed that the relative improvement due to stabilization in response to deformation is more significant than the strength increase. With the addition of small amount of lime, many soils behave brittle and exhibits small deformation in the elastic state but very low strength. Balasubramaniam et al., (1989) studied the strength and deformation behavior of lime treated soft Bangkok clay using undrained triaxial compression with a curing period of one month. They found that the lime content of 2.5% is not effective. The increase in lime content from 2.5% to 12.5% did not increase the shear strength significantly after first month but after two months lime content of 5 to 15% resulted strength gain of about 10 times the strength of untreated ground. Also, varying lime content did not affect the angle of friction. Benjamin (1990) studied the behavior of the lime treated Bangkok clay through the laboratory test. The soil with 4.3% organic content and lightly overconsolidated clay was used. Quick lime powder was used to stabilize the soft clay ground. The lime content varied from 2.5% to 12% and curing time was up to 2 months but he found 10% the lime content is the optimum lime content. The test results showed the increase in strength and decrease in compressibility co- efficient.

The chemico-pile soil improvement method has been used in a full-scale instrumented embankment project located at Nong Ngo Hao site near Bangkok. This site is a part of second Bangkok International Airport (SBIA) project, located in Samutprakan Province, 30 km east of the Bangkok Metropolis. Both laboratory and field investigations were performed to evaluate the effectiveness of lime column on soft soil. Soil samples for laboratory testing were collected from one-third and half diagonal distance between two columns. The experimental program was utilized to evaluate the improved soils conditions due to chemico-pile. In-situ Cone Penetration Tests (CPT) were performed to evaluate the reduction in compressibility and increase in shear strengths of soils. Based on the improved soils parameters, the improved embankment behavior was predicted using the Finite Element Program PLAXIS. The objective of numerical modeling was to predict the settlement, lateral deformation, and pore pressure of the Nong Ngu Hao Test embankment. The objective of the current paper is to: (i) Investigate the effect of chemico- pile on different soil parameters and evaluate performance of embankment built on chemico-pile improved soils. (ii) Numerical modeling of Nong Ngo Hao test embankment site improved with the chemico-pile, using PLAXIS, a finite element program, to predict the strength and deformation characteristics of the improved site and finally (iii) Compare the predicted results with the actual observed field data.

4.2 Site Condition and Soil Profile

The Nong Ngu Hao test embankment site is located at Samutprakan Province. The project area is about 4 km x 8 km. The proposed second Bangkok International Airport site is locatsed within the southern part of the Chao Phraya Basin, formed by fault block tectonics during tertiary period. The depression was filled with three major kinds of depositing environments, namely: alluvial fan type, alluvial flood plain environment, and deltaic environment during tertiary and quaternary geological period. The Chao Phraya Delta is formed by accumulation of suspended sediments. It consists predominantly of clay, silt and fine-grained sand. The depositing environment of the shallow subsoil is likely to be a river plain or a delta with occasional shallow marine clay. As per recent trends, the sea level rises up and deposits fine particles on the previous coarse particle layer. This leads to the typical soil profile in Bangkok area, which consists of alternating clay and sand deposits. There are lenses of various layer types while the very soft clay and weathered clay form the upper most part of soil profile. The site investigations consisted of two soil test borings (BH-1 and BH-2) of 30m depth and undisturbed samples were collected at each 1.5 m interval in the soft clay layer. The general soil properties of embankment site are presented in Figure 4.2

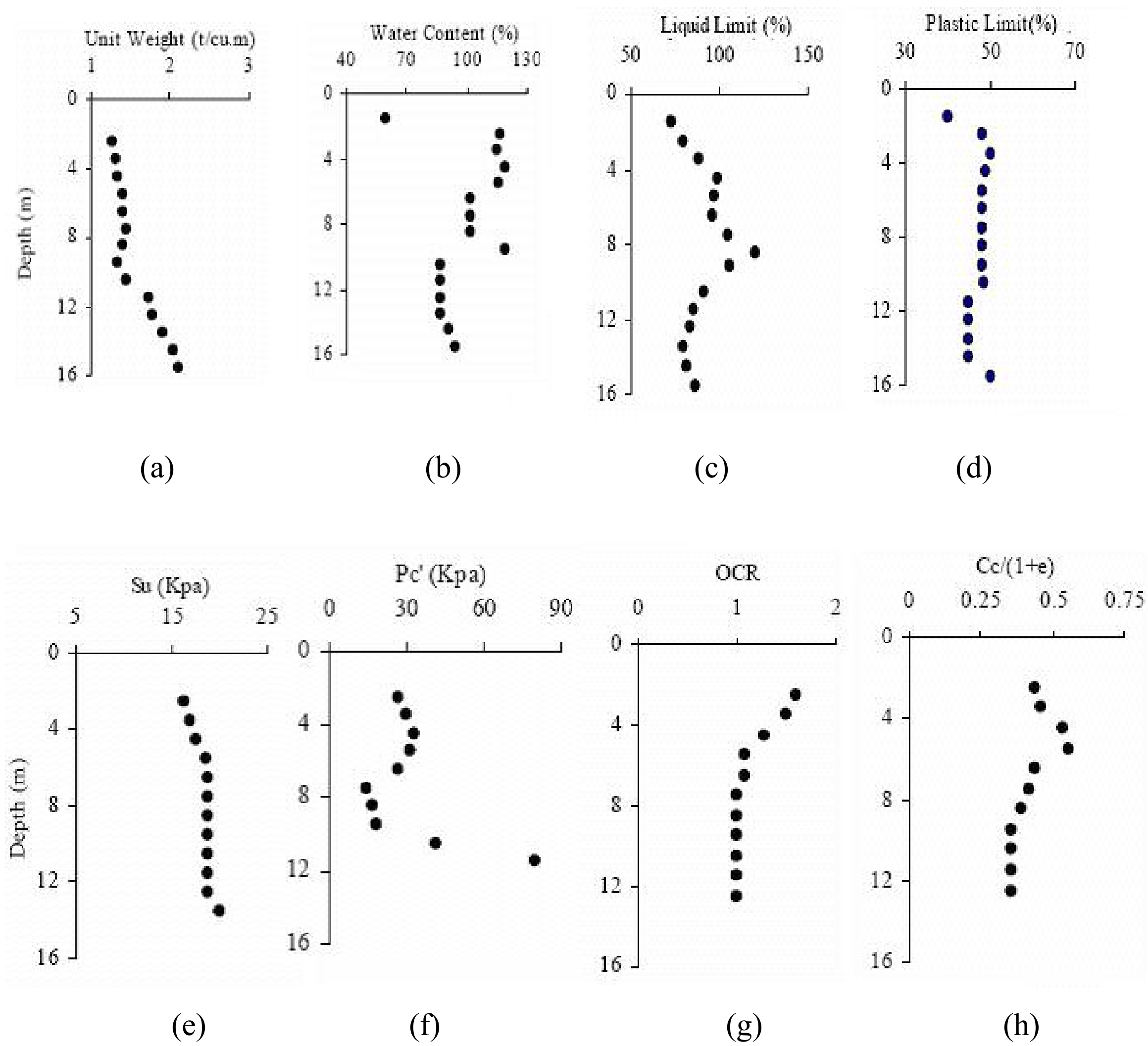


Figure 4.2 Generalized Soil Properties of Embankment Site (a) Variation of Unit Weight (b) Variation of Water Content (c) Variation of Liquid Limit (d) Variation of Plastic Limit (e) Variation of S_u (f) Variation of P'_c (g) Variation of OCR (h) Variation of Compression Ratio

The Standard Penetration Tests (SPT) were also performed in the stiff clay and sand layers. Based on the site investigations, a general soil profile for the site was developed. The soil profile is relatively uniform consisting of a thin weathered clay crust overlying a layer of soft Bangkok clay of approximately 12m thick. The natural water content of soft and very soft clay layer is between 55% and 62% while the liquid limit ranges from 90% to 114%. The undrained shear strength ranges from 14 kPa to 18 kPa. A stiff clay layer underlies the soft clay and extends to a depth of 20 to 24 m below the ground surface.

4.3 Concepts of Lime Stabilization Technique

4.3.1 Properties and Types of Lime

The Lime used for soil stabilization are in the form of quicklime, CaO or hydrated lime, $Ca(OH)_2$. Commonly, lime refers to as products of calcite and dolomite limestone. Hence, lime can be classified into two groups, calcite lime and dolomitic lime. The formation of calcitic lime involves the following chemical processes:

$$CaCO3 + heat \text{ -----> } CaO + CO2 \qquad (4.1)$$

The process is endothermic in nature as heat is required to dissolute CaCO3 (Kezdi, 1979). Calcitic quicklime can easily hydrated according to the following equation:

$$CaO + H2O \text{ -----> } Ca(OH)2 + 65.3\ kJ/mol \qquad (4.2)$$

Hydration of dolomitic lime follows the reactions:

$$CaO + MgO + H2O \text{ -----> } Ca(OH)2 + MgO \qquad (4.3)$$

Normally, finely pulverized quicklime is used for clays with moderately high water content (Broms & Boman, 1977). For clays with high water, as in the case of Nong Ngo Hao site, specially prepared chemico-lime (quicklime with additives) was used for soil stabilization to support embankment.

4.3.2 Mechanism of Lime Stabilization

Lime treated soft clay gains strength mainly due to three reactions namely, consolidation effects or hydration, ion exchange and pozzolanic reaction. Other mechanism such as carbonation causes minor strength increases, which is normally disregarded.

4.3.2.1 Consolidation or Hydration

A large amount of heat is released when quicklime (CaO) is mixed with clay due to hydration of quicklime with pore water of soil. The increase in temperature can at times, be so high that the water starts to boil (Broms, 1984). An immediate reduction of natural water contents occur when quick lime is mixed with cohesive soil as water is consumed in hydration process. Moreover, a considerable amount of pore water evaporates due to release of heat during the reaction.

$CaO + H_2O \text{ -----> } Ca(OH)_2$ + 15.6 Kcal. /mol; (280 Kcal/Kg) (4.4)

The calcium hydroxide $Ca(OH)_2$ (from the hydration of quicklime) or application of calcium hydroxide as the stabilizer, dissociates into pore water, increasing the electrolytic concentration and the pH of the pore water. This further dissolves the SiO_2 and Al_2O_3 from the clay particles.

$$Ca(OH)_2 \text{ -----> } Ca^{++} + 2(OH)^{-} \quad (4.5)$$

The dissociation will result in ion exchange and flocculation as well as pozzolanic reaction.

4.3.2.2 Ion Exchange and Flocculation

When the lime is mixed with clay, sodium or other cations absorbed to the clay mineral surfaces are exchanged with calcium. The change in cation affects the complex structural component of clay mineral. Calcium hydroxide is transformed again due to presence of the carbonic acid (which is in soil) due to reaction of carbon dioxide of the air in soil and water. The reaction results in the dissociation of the lime into Ca^{++} and $(OH)^-$ which modifies the electrical surface forces of the clay minerals.

$$Ca^{++} + \text{Clay} \dashrightarrow Ca^{++} \text{ Exchange with monovalent ions } (K+, Na+) \quad (4.6)$$

As a result, clay particles are bonded with each other and results in increase in the shear strength.

4.3.2.3 Pozzolanic Reaction Effects

The shear strength of the stabilized soil gradually increases with time due to pozzolanic reactions. Calcium ions (Ca^{++}) from the stabilizer continue to react with SiO2 and Al2O3 in the clay for a long time and forms CaO.SiO2.H2O causing the clay to gain strength. The reaction is called pozzolanic reaction.

$$Ca^{++} + 2(OH)^- + SiO2 \text{ -----> } CSH \quad (4.7)$$

$$Ca^{++} + 2(OH)^- + SiO2 + Al2O3 \text{ -----> } CSH \quad (4.8)$$

The gels of calcium silicates (and/or lime silicates) induce cementing effects on the soft soil particles. Chemico-lime piles have considerable strength and therefore reinforce the soil in addition with changing the natural properties of soil.

4.3.2.4 Effects of Chemico-Pile on Adjacent Soil

Chemico-pile has specific hydraulic properties. The strength of the pile generally increases linearly with time and joins the surrounding soil with pile bonds through pozzolanic reactions. The improved soil by chemico-pile method is considered as composite soil structure consisting of chemico-pile and intermediate soil between piles. The following relationship is acceptable for strength and deformation modulus of chemico-pile and surrounding soils:

Chemico-pile > Surrounding soil > Middle of soil between piles

4.4 Field Instrumentations of Test Embankment

The plan and sectional views of instrumentation for the test embankment are presented in Figure 4.3. The base dimension of embankment was 15m x 15m and the final height of preloading was 2m. The Chemico-pile of 0.4m diameter and 16m long were installed at 1.2 m (3d) intervals. The top 1m of test sections were covered with the natural soil excavated from the vicinity and mixed with Chemico Lime. This top layer is called "Chemicolizer" treatment, which is the general surface lime stabilization method. The experimental program and in-situ CPT tests of the improved ground were carried out at two locations: at mid and one-third diagonal distance between two columns. The field monitoring of embankment was performed for 42 days after the embankment was constructed.

The embankment was instrumented with piezometer, surface settlement gauges, subsurface settlement gauges, inclinometer and observation wells. The key measurements include pore pressure, settlement of ground surface, deep settlements of the sub-soils, lateral movement and ground water level.

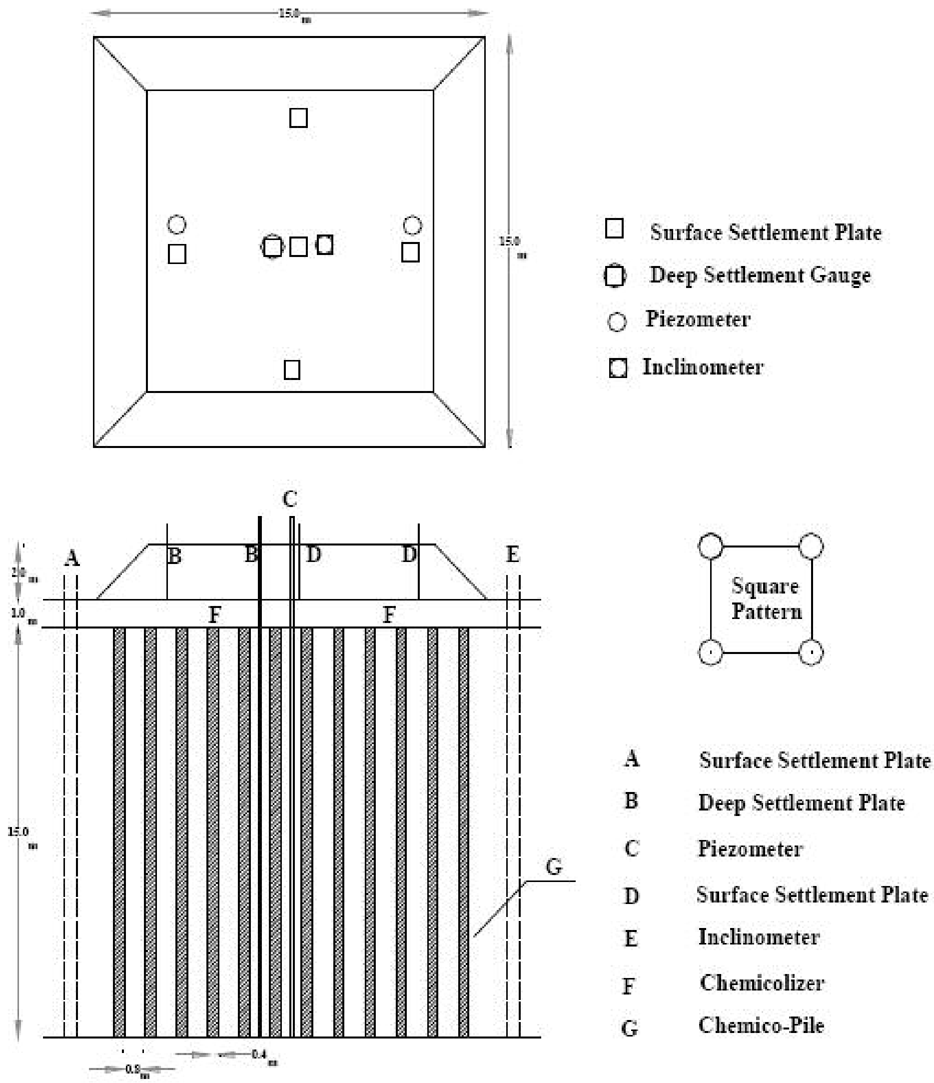


Figure 4.3 Embankment Instrumentation Plan

The field monitoring instruments are summarized in Table 4.1. Curing period of the test embankment with chemico-pile was 90 days. After 90 days, the embankment load was placed and monitoring key properties was done for 42 days. The total pore pressures were monitored at depth of 3, 7 and 14 m from the ground surface. The surface settlement plates were installed at the top of completed embankment to monitor the post construction surface settlement.

Table 4.1 Field Monitoring Instruments

Number of Instruments	Instrument Type	Properties
2	Piezometers	Excess Pore Pressure
5	Surface Settlement Gauges	Surface Settlement
1	Subsurface Settlement Gauges	Subsurface Settlement
2	Inclinometer	Lateral Movements
1	Observation Well	Ground Water Level

The surface settlement plates are located at the distances of 1m and 6 m from the center of the embankment. In order to monitor the settlements at deeper depths, one deep settlement plate was installed. Settlements are measured daily by leveling with reference to benchmark. Two inclinometers were installed at the toe of the embankment to measure the lateral deformation of the test embankment.

4.5 Finite Element Method

In the finite element method, the actual continuum or the body (Pile Supported Embankment model in this case) is represented as an assemblage of subdivided interconnected finite elements. The subdivided elements are linked at specified joints called nodes or nodal points. It is presumed that the field variable (i.e., displacement, stress, temperature, pressure, or velocity) within the interconnected subdivided element can be approximated by a simple function. These approximating simple functions (also called as interpolation models) are defined in terms of field variables at the nodes. When the equilibrium equations (field equations) are compiled, the unknown will be the nodal values of the field variable. The nodal values of the field variables are computed by solving the field equations, which is normally in matrix form. From the computed nodal values and with the aid of approximating functions field variables are determined. The model execution of finite element process is done in methodical procedure. The general procedures of the steps involved in the process are stated as (Rao, 2005):

- Discretization of the model: The primary step in any finite element method in which the body or the model is subdivided into finite elements. The type, size, number of such subdivided elements is a user choice.
- Selection of a proper interpolation or displacement model: Considering the fact that precise predictions of displacement solution cannot be done for complex models under loads, a suitable assumption can be made which satisfy certain convergence requirements (generally interpolation model is considered in form of polynomial).
- Derivation of element stiffness and load vectors: Using the equilibrium conditions or suitable variational principle with the aid of suitable assumed displacement model, the stiffness matrix [k] and load vector ‘p’ for each element are derived.

• Assemble of global stiffness matrix: Since the model is divided into several finite elements, the individual stiffness matrix [k] and the load vector ‘p’ are to be arranged in a suitable manner. The overall equilibrium equations is formulated as:

$$[K^{'}]\acute{Ø} = P^{'} \tag{4.9}$$

Where $[K^{'}]$ is the assembled stiffness matrix, $\acute{Ø}$ is the vector of the nodal displacements and $P^{'}$ is the vector of nodal forces for the complete model.

• Computations of unknown nodal displacements: Incorporating the boundary conditions the equilibrium equation (4.1) can be rewritten as

$$[K]\acute{Ø} = P \tag{4.10}$$

Nodal displacement vector ($\acute{Ø}$) is easily computed of for a linear problem however, for non-linear problems, the value is computed in a sequence of steps. Modification of stiffness matrix [K] and load vector P are done for each sequence.

• Computation of stress and strain: From the computed nodal displacements, elemental stress and strains can be computed.

The user-friendly window based finite element models are frequently used now a days. However, the computed results depends on the input parameters which include the boundary conditions, materials type, material behavior, type of model and its parameters. Therefore, the material model should be selected carefully based on their behavior, loading path, stress level corresponding to the considered case of analysis. The finite element program PLAXIS developed by PLAXIS BV has been adopted for the numerical analysis and modeling of embankment over soft clay improved with chemico-pile. The chemico-pile embankment is modeled as a two dimensional plain strain model. The finite element mesh for soils layers has been generated in accordance with the actual soil profile which consists of 4 layers. The 15 nodded triangle elements were used for both subsurface and ground surface including the embankment. The lime-piles of 0.4 m diameter and 16 m lengths have been simulated as 5 noded beam (structural) elements. The interface elements have been considered along the chemico-piles to take care of the soil structure interaction problem and to introduce reduction factors, if needed. The cross-section of the numerical model is presented in Figure 4.4. The bottom boundary is fixed and the two sides are free to move in vertical direction.

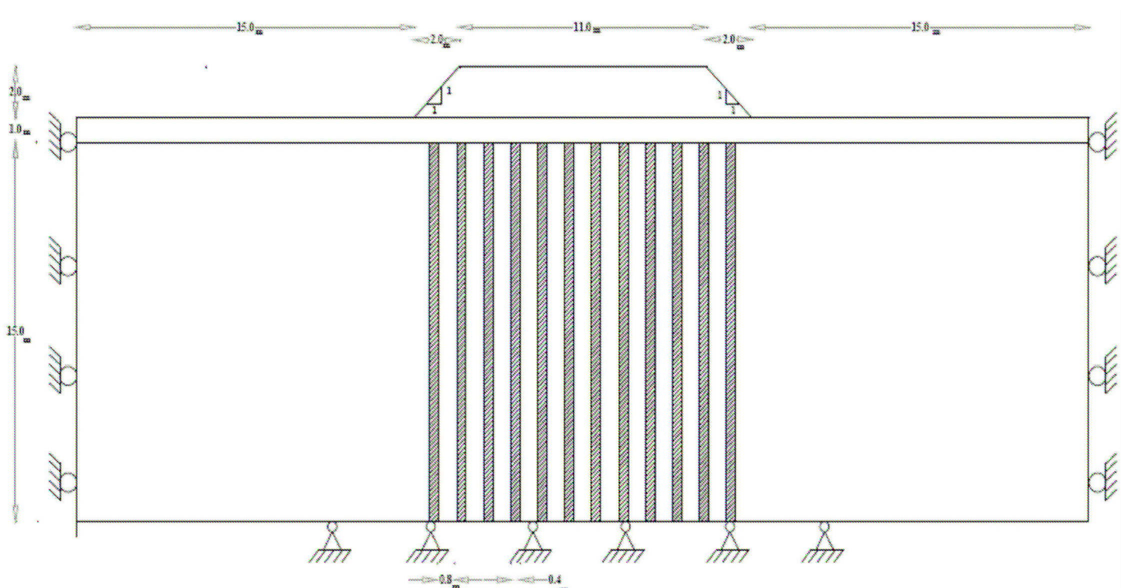


Figure 4.4 Embankment Cross-Section and Boundary Conditions

The finite element mesh for soils layers has been generated in accordance with the actual four soil layers. The soft soil below the embankment, outside the embankment and embankment fill soil were modeled as elastic perfectly plastic materials. Considering the sensitivity of soil parameters and available soil parameters, the Mohr Coulomb model (MC) has been used for all the soil layers. The soil parameters determined from triaxial tests and in-situ tests were used as input parameters. Based on the undrained shear strength of site soils, Young's modulus (E) of Nong Ngu Hao clay soil was determined using the following co-relations:

$$E = \alpha .Su$$

Where, S_u = undrained shear strength

α = co-efficient (for Nong Ngu Hao clay, 75 -100)

For numerical modeling, co-efficient α has been considered as 100. Based on the young's modulus, the shear modulus G, which is the stiffness modulus in Mohr Coulomb model, is determined using the following c-relation:

$$G = E / \{2(1+\upsilon)\}$$

Where, υ = Poisson's ratio

The poison's ratio for the soft clayey soil has been taken as 0.30 to 0.35 and the dilatancy angle ($\psi = \varphi - 30^o$) was taken as zero for all soils, as the friction angle for all four site soils are less than 30^o. The soil parameters for M-C model have been tabulated in Table 2. The ground water table for the site has been considered at the ground surface. Consolidation analyses have been performed to simulate the excess pore water pressure with time and therefore, the undrained soil parameters have been considered for the present study.

4.6 Results and Discussions

Improvement effect of chemico-lime piles on Clay was observed by both laboratory and in-situ field Cone Penetration Test (CPT). Samples for the laboratory tests were collected at two locations: at mid and one third diagonal distance between two columns. The in-situ CPT tests were performed at the lime column location in soft clay. The natural water content and Atterberg Limit of improved soil were determined to assess the degree of stabilization. The UU triaxial tests were performed for undrained shear strength; Dutch Cone Penetration Test (CPT) and consolidation test were performed for evaluation of strength and compressibility characteristics of the soft Nong Ngu Hao clay improved with Chemico pile. The laboratory tests were performed at different curing times: 7 days, 15 days, 30 days, 60 days, and 90 days after the instillation of the chemico pile. The test results for 0 days and 90 will be presented for simplifications. The experimental results showed that the natural water content of the improved soil at each depth along the column is decreased with time and tends to be constant after 30 days of curing period. The reduction of water content is in the order of 20% - 30%. The same behavior was observed for plasticity index with a decrease of 10-15%. The reduction of water content and plasticity index is more close to chemico pile compared to the reduction of water content at 1/3 distance from the chemico pile.

Significant change of void ratio was observed due to the dewatering effects of the ground from the hydration of quick lime. With the result of this, the compressibility ratio will go down and the settlement value will be less than the untreated ground. Maximum reduction in compressibility ratio has been observed to decrease up to 55% between 5m and 6m depth, which is very soft clay. Hence, it can be argued that the chemico pile has maximum effect on the very soft clay. Based on the experimental program, the undrained shear strength decreases with time up to the first 30 days and then it starts building up again. The average strength after 30 days is about 40% of the initial strength. In some cases, the strength after 90 days of curing time is less than the initial strength but it tends to increase with time, which may be due to effect of reconsolidation. The decrease in strength is mainly due to the disturbance of surrounding soil during the excavation of chemico pile. The very high sensitivity of Nong Ngu Hao clay is the main reason for disturbance which leads to the decreases in strength.

One of the most important controlling factors for the performance of reinforced earth is simulation of soil-structure interaction. The interface elements have been considered along the chemico-pile to take care of the soil structure interaction problem and to introduce reduction factors, if needed. The flat element (Zienkiewicz, 1970) has been used for soil- structure interface. In formulating the stress-strain behavior at interface, the thickness, t, of the interface element is necessary.

This is only a non-physical parameter that can be assumed as 0.1 to 0.01 times of the length of the corresponding interface element (Desai et al., 1984). In PLAXIS program, the stress-strain behavior at soil-structure interface is simulated by elastic-perfectly plastic interface model. The parameters for embankment, effective stress parameters, subsoil parameters and materials properties are summarized in Table 4.2, Table 4.3, Table 4.4 and Table 4.5, respectively.

Table 4.2 Selected Parameters for Embankment in PLAXIS Analysis

Embankment	Model	ν	G (kPa)	E (kPa)	c'(kPa)	φ' (Degree)
2m Height	M-C	0.3	3076	8000	1	30

Table 4.3 Selected Effective Parameters for FEM Analysis

Depth of Soil Layer	Model	ν	G (kPa)	E (kPa)	c'(kPa)	φ' (Degree)
16.0 -11.5	M-C	0.3	7000	2900	5	23
11.5- 8.5	M-C	0.3	1500	2300	2.5	23
8.5 - 1.5	M-C	0.3	700	2100	4	23
1.5 - 0	M-C	0.3	7692.3	20000	200	23

Table 4.4 Selected Parameters for Sub Soil Parameters for FEM Analysis

Depth of Soil Layer	Model	ν	G (kPa)	E (kPa)	c (kPa)	φ (Degree)
16.0 -11.5	M-C	0.3	7000	2900	29	0
11.5- 8.5	M-C	0.3	1500	2300	23	0
8.5 - 1.5	M-C	0.3	700	2100	21	0
1.5 - 0	M-C	0.3	7692.3	20000	200	0

Table 4.5 Material Properties for FEM Analysis

Material Set	Drainage Conditions	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	k_x (m/d)	k_y (m/d)	R_{inter}
Medium Clay	Undrained	18	18	0.0007	0.0006	1
Soft Clay	Undrained	17	17	0.0009	0.009	1
Very Soft Clay	Undrained	16	16	0.005	0.005	1
Chemicolizer	Undrained	15	22	0.009	0.0009	1

The model parameters at soil-structures interface can be generated from that of the soil using the interaction co-efficient, (Ri) defined as the ratio of the shear strength of soil-structure interface to the corresponding shear strength of the soil (Vermeer & Brinkgreve, 1995). The 2m high embankment was constructed by placing the fill material layers in two lifts in a period of four days. In finite element analyses, the embankment loading of 1m was applied instantaneously after two days and second 1 m load was applied instantaneously after four days. Therefore, initially, the results obtained from numerical modeling are expected to vary from actual field data.

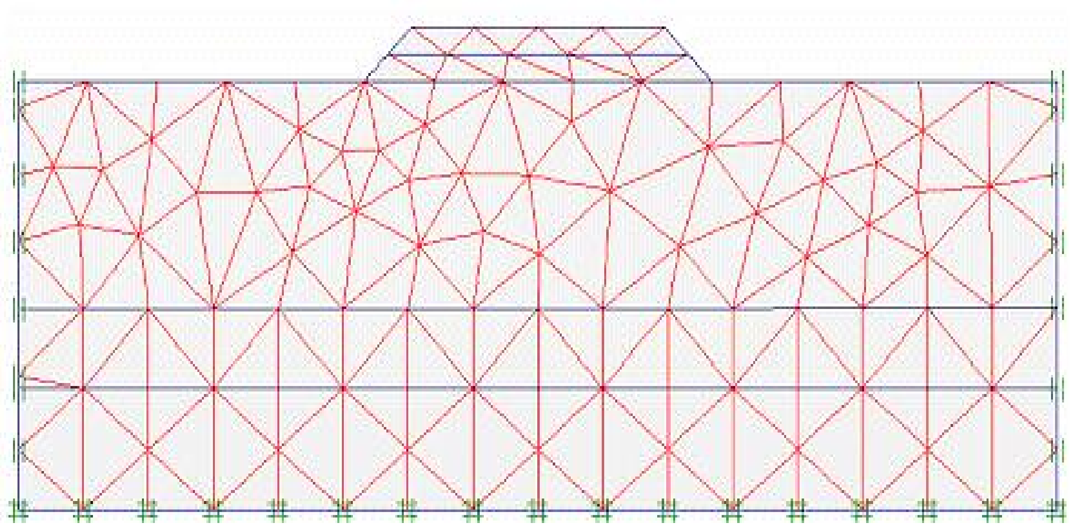
4.7 Embankment Modeling- PLAXIS

PLAXIS version 8 has been used for the current analysis for calibrating pile-supported embankment. The parameters for material model for calibrating are taken from the report "Prediction Versus Performance of the Nong Ngu Hao Site Improved Using the Chemico Pile Method" by Hossain (1997). The model parameters for the foundation soil, embankment properties and pile properties are tabulated in Table 4.2.

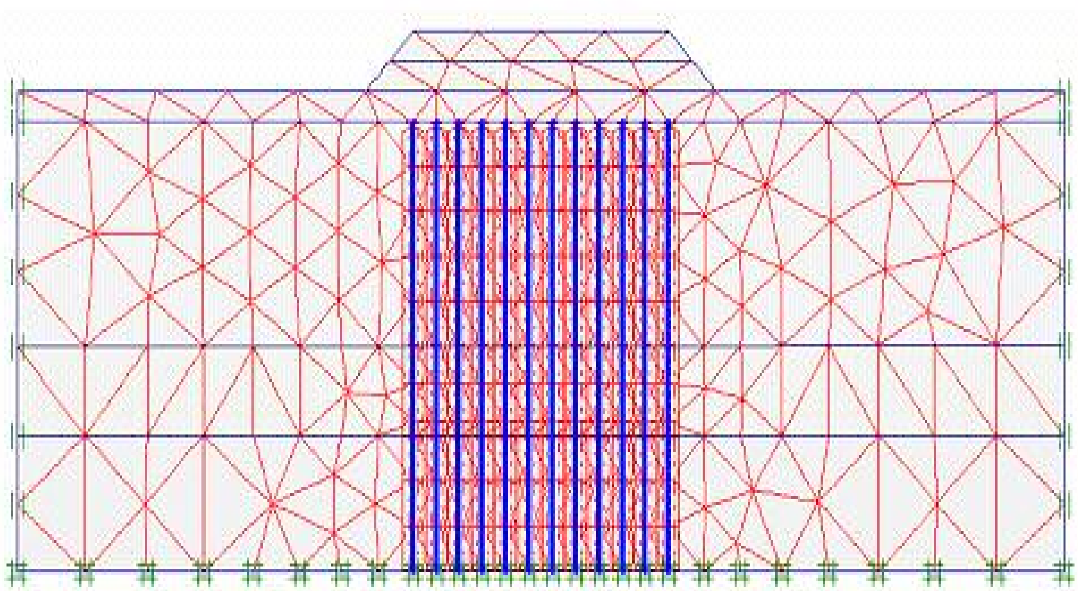
4.7.1 Mesh Generation

Model with the boundary conditions and triangular mesh is presented in Figure 4.5 (a) and Figure 4.5 (b) for untreated and treated ground respectively. A 15-node triangle mesh is an accurate element that provides high quality stress results for complex non-linear problems. A 15-node element provides fourth order interpolation for displacement and the numerical integration involves 12 stress points.

PLAXIS allows for a fully automatic generation of finite element meshes. The mesh generator (developed by Sepra) develops triangular, unstructured mesh based on robust triangulation procedure. These meshes looks disorderly, but numerical performance of such meshes are better than the structured meshes (PLAXIS, 2000).



(a)



(b)

Figure 4.5 FEM Mesh PLAXIS (a) Untreated (b) Treated

4.7.2 Water Pressure Generation

Phreatic level is considered at the ground surface for the current model. Water pressure in PLAXIS can be generated by phreatic level or by means of complex groundwater flow calculation. Figure 4.6(a) and Figure 4.6(b) presents the water pressure contours and the shading for untreated ground.

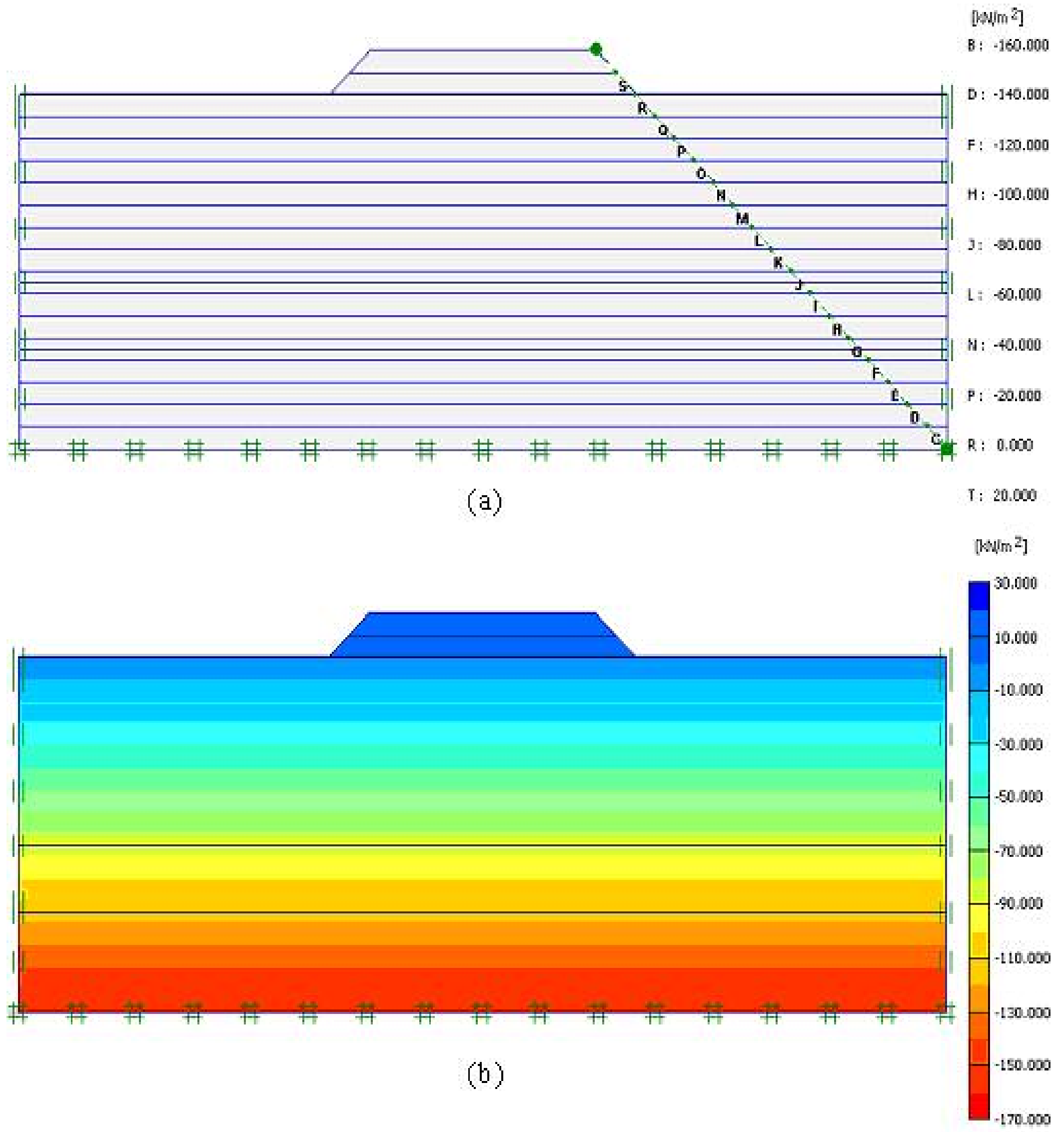
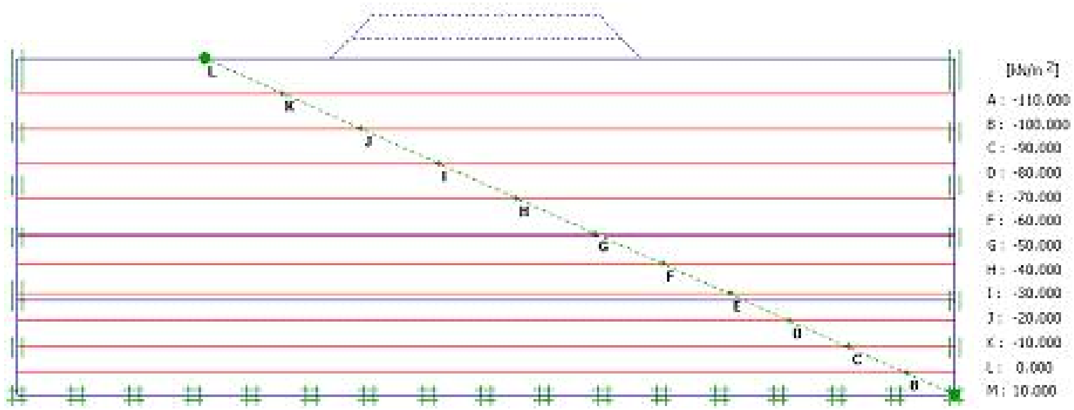


Figure 4.6 FEM Plot of Water Pressure of 160 kN/m^2
(a) Contour Lines (b) Shading

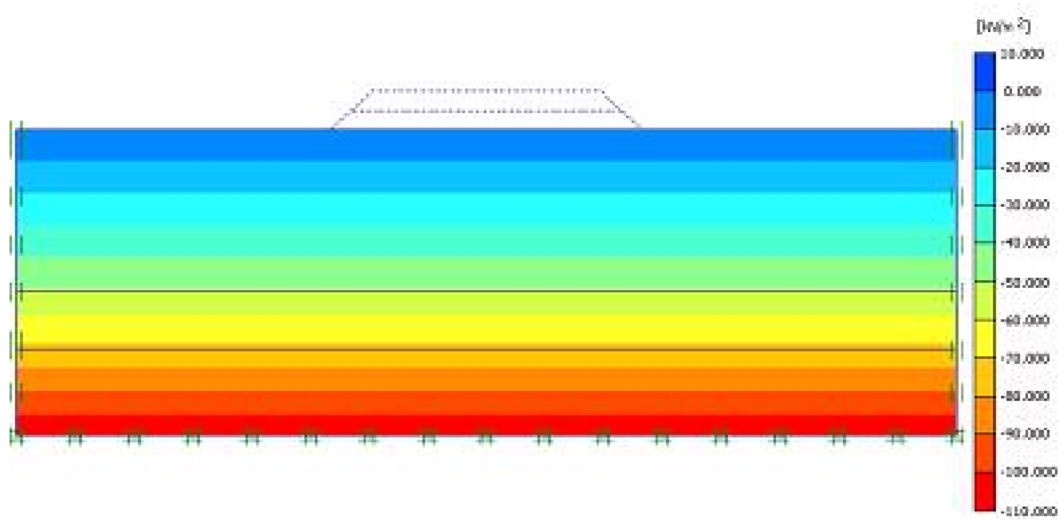
Generation of water pressure by phreatic level being quick and straightforward as compared to complex but realistic, in present model the water pressure is generated by phreatic level. It is observed from the model outputs that the water pressure for the treated and untreated ground being same.

4.7.3 Initial Stress Generation

The history of soil formation and the weight of the materials influence the initial stress in the soil body. The initial stress output is presented in Figure 4.7.



(a)



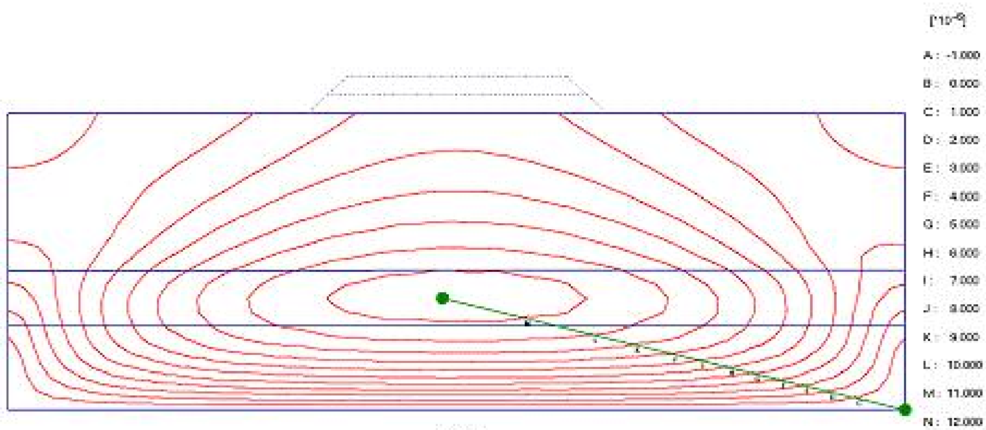
(b)

Figure 4.7 FEM Plot of Initial Stress (a) Contour Lines (b) Shading

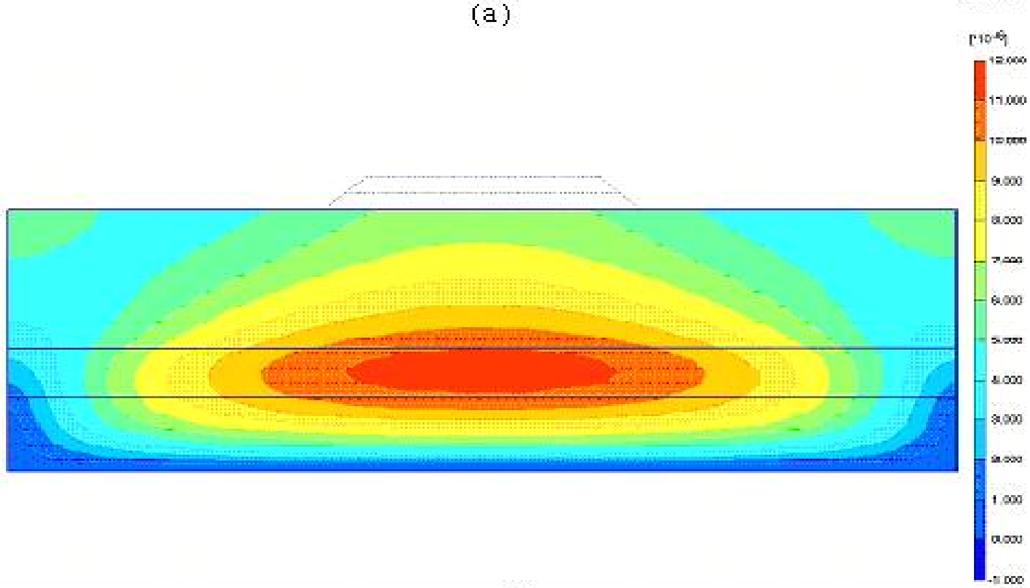
The stress state is characterized by initial vertical effective stress (σ'v,o) and initial horizontal effective stress (σ'h,o) which is interrelated by coefficient of lateral earth pressure, Ko as σ'h,o = Ko σ'v,o.

4.7.4 Gravity Loading

In this computational stage, the existing excess pore pressure, which was generated will remain but no new excess pore pressure will be generated. Figure 4.8 and Figure 4.9 presents the outputs of gravity loading for untreated and treated ground.



(a)



(b)

Figure 4.8 FEM Plot of Gravity Loading for Untreated Ground
(a) Contour lines (b) Shading

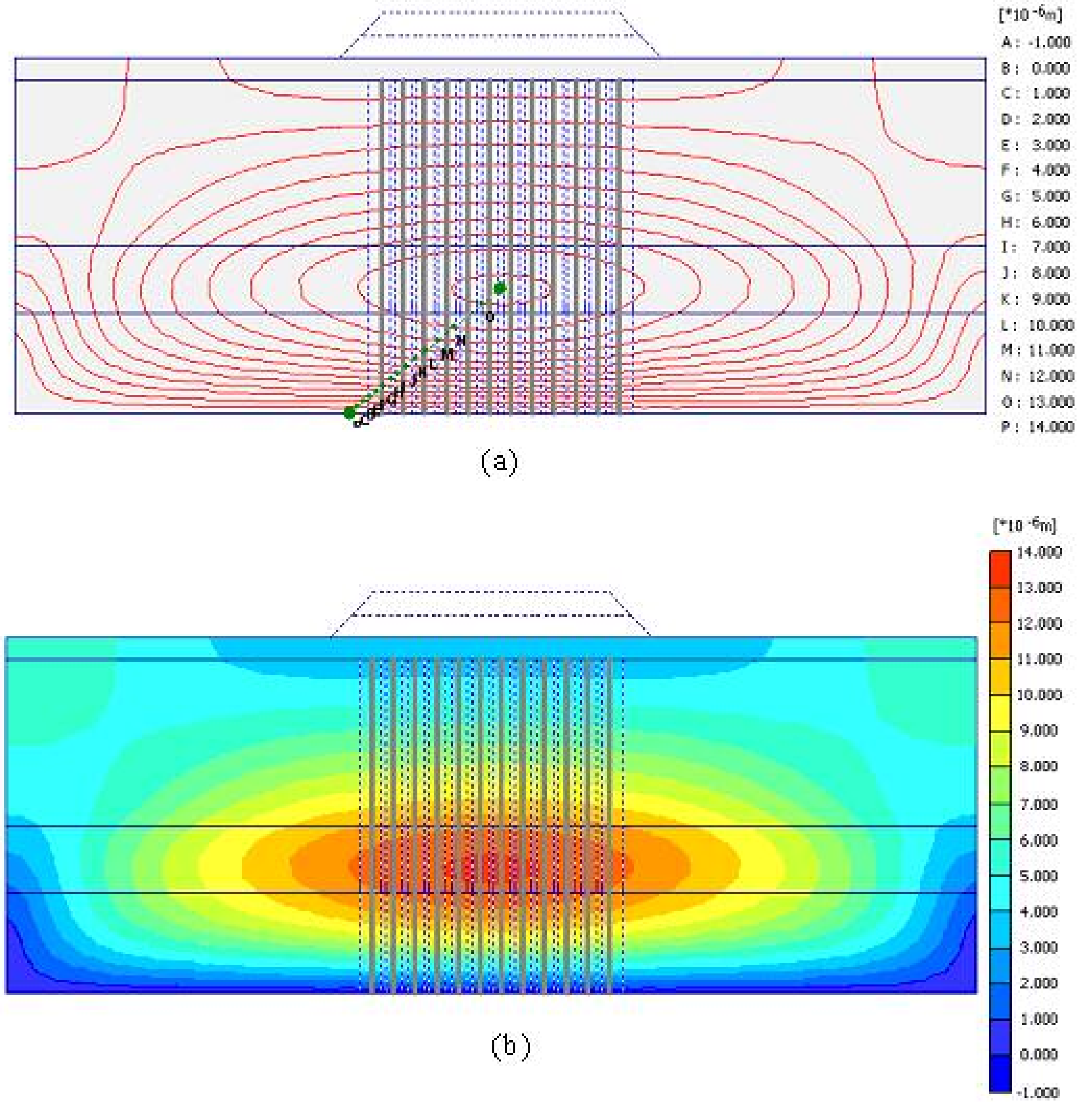


Figure 4.9 FEM Plot of Gravity Loading for Treated Ground
(a) Contour Lines (b) Shading

4.7.5 Structural Element

Activating the structural element (piles in the present treated model) is done in this stage. The deformation output is presented in the Figure 4.10. The type of element for structural elements and interfaces is compatible with the soil element type.

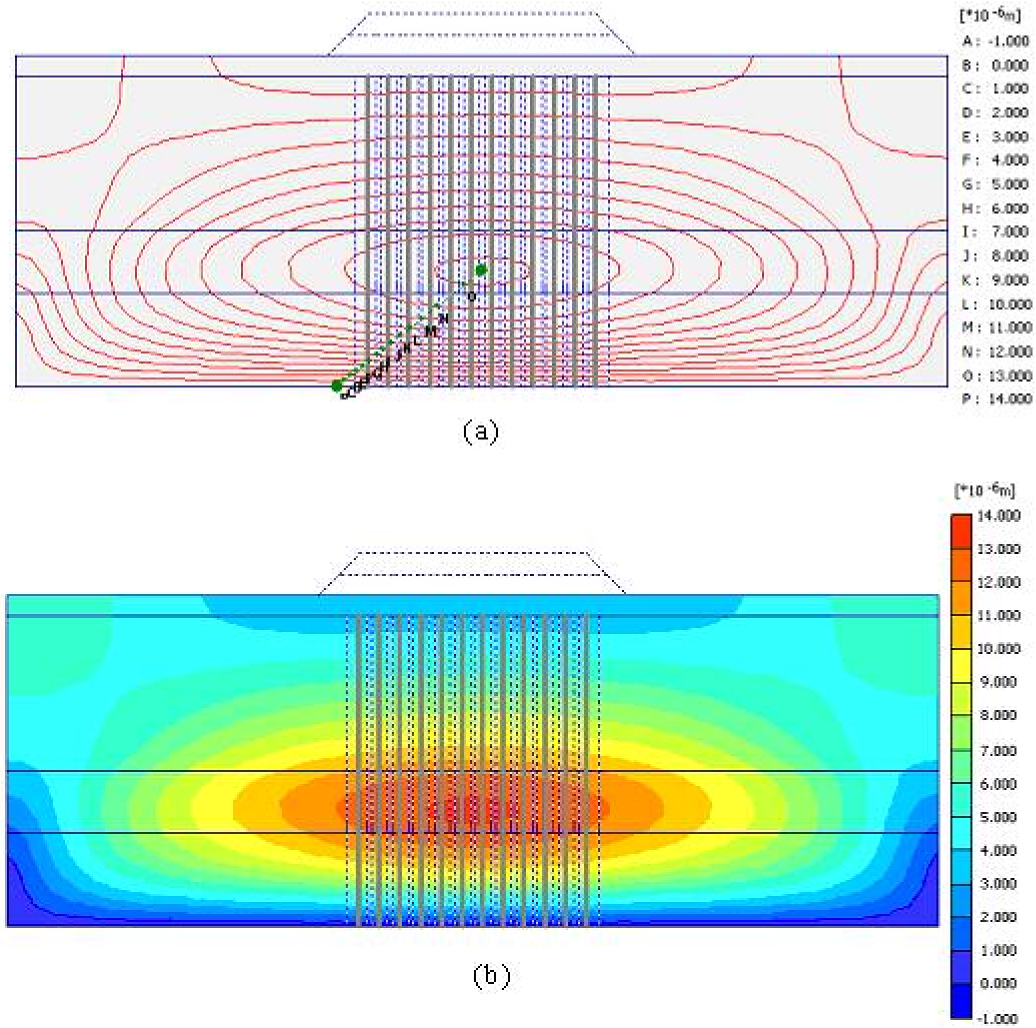


Figure 4.10 FEM Plot of Extreme Total Displacement due to Structural Member of $13.17*10^{-6}$m (a) Contour Lines (b) Shading

4.7.6 Stage Embankment construction (1m)

Deformation analyses for embankment construction are done in stages of one meter each. In first stage, one meter of embankment is constructed within two days. In numerical modeling, construction load is assumed as instantaneous loading. The stage construction is the most important type of loading. The total deformation for the first lift of the embankment is presented in Figure 4.11 and Figure 4.12 for untreated and treated ground respectively. In the FEM analysis it is possible to change the geometry and load configuration by deactivating or reactivating loads, soil clusters or any structural objects as created in the initial geometry generation. Staged construction analysis can be performed in both plastic calculation and consolidation analysis.

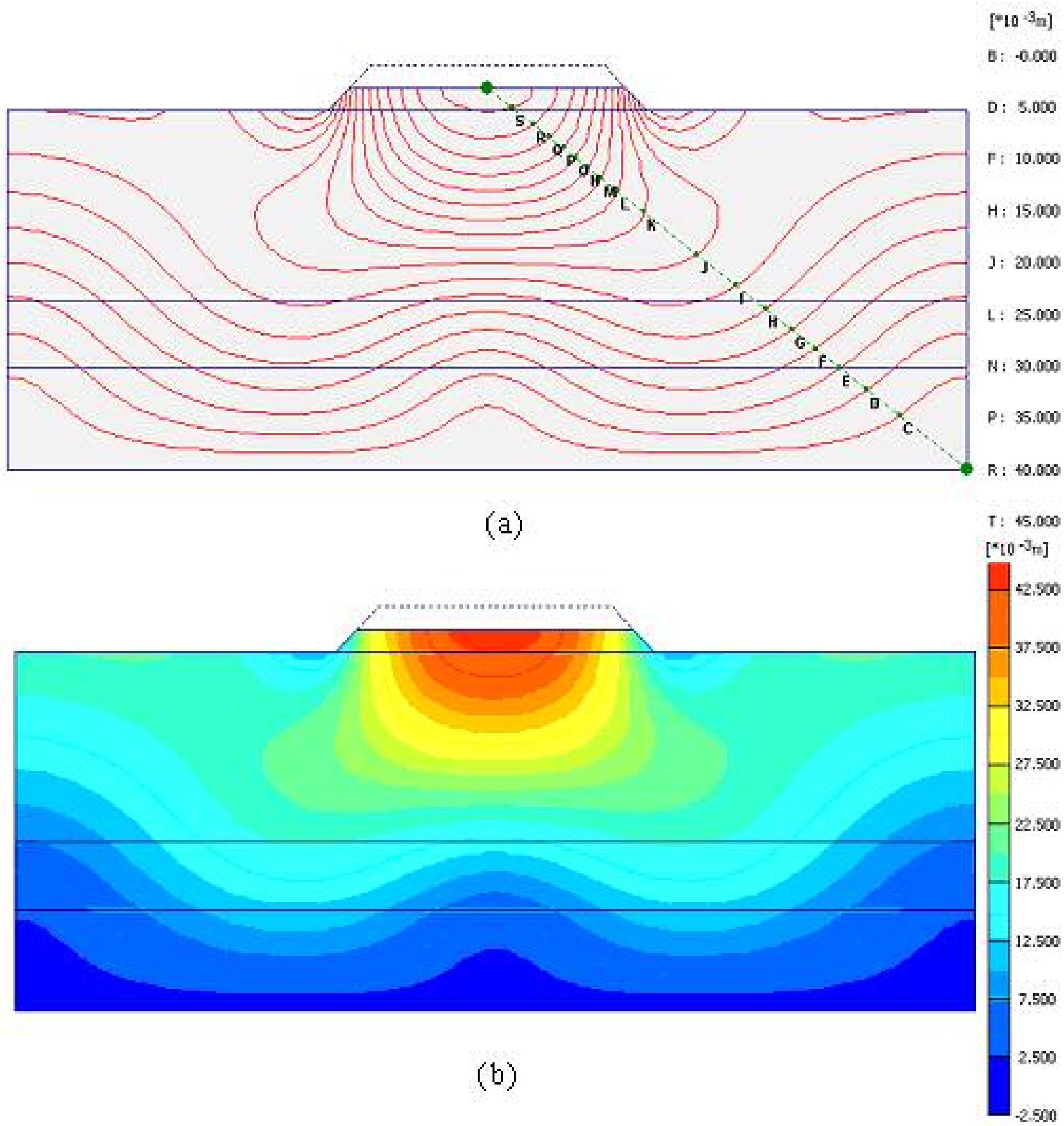


Figure 4.11 FEM Plot of Extreme Total Displacement for 1m Lift of Embankment (Untreated) 44.34*10^{-3}m (a) Contour Lines (b) Shading

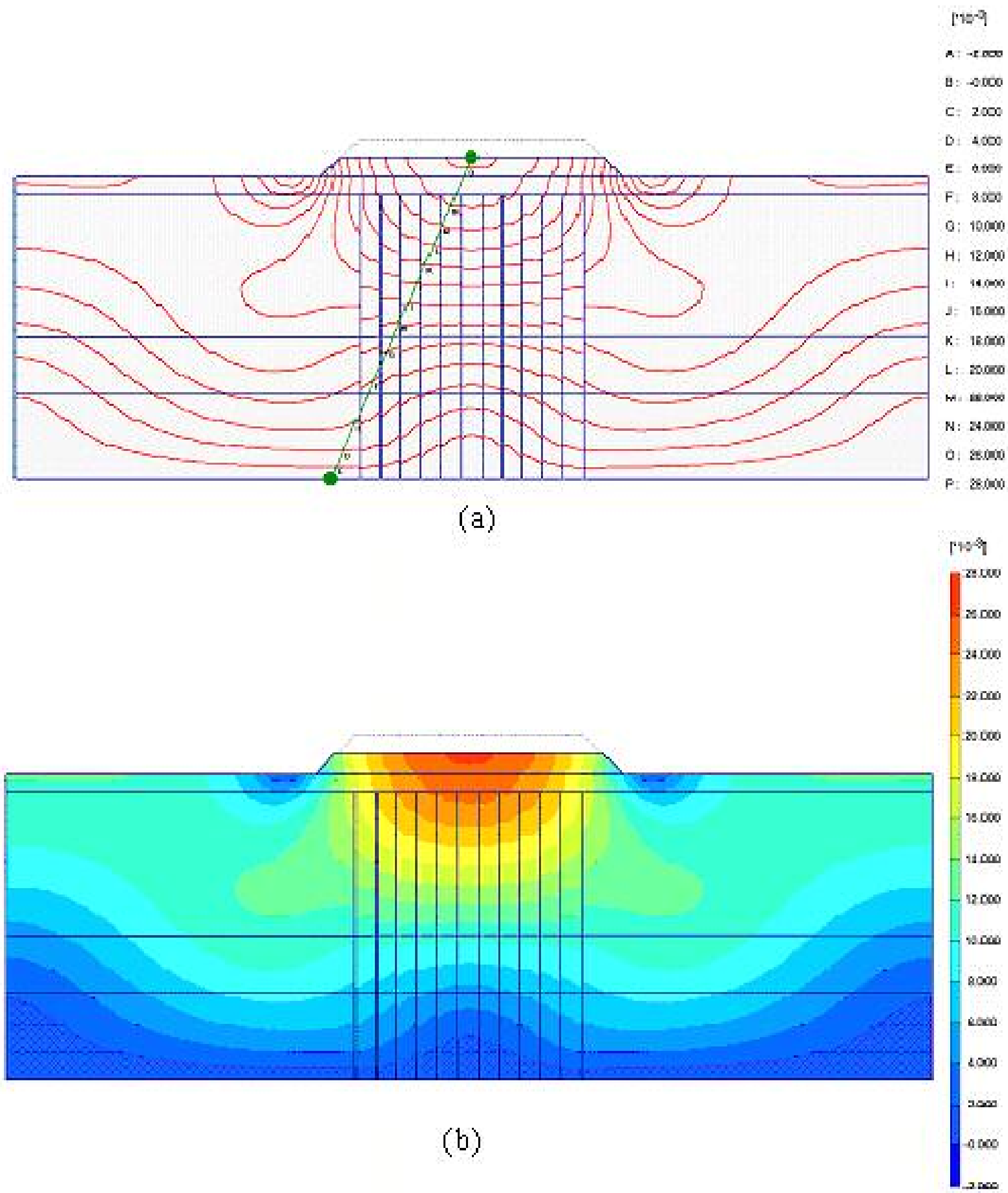


Figure 4.12 FEM Plot of Extreme Total Displacement for 1m Lift of Embankment (Treated) 26.31×10^{-3}m (a) Contour Lines (b) Shading

4.7.7 Stage Embankment construction (2m)

The embankment construction is completed in the second phase with 1mt of fill placement. The deformation analysis output for both untreated and treated is presented in Figure 4.13 and Figure 4.14. An element when reactivated in a calculation phase where the material data has been set to undrained, then the element will behaves as a drained materials. This phenomenon allows development of effective stress due to self-weight in the activated soil.

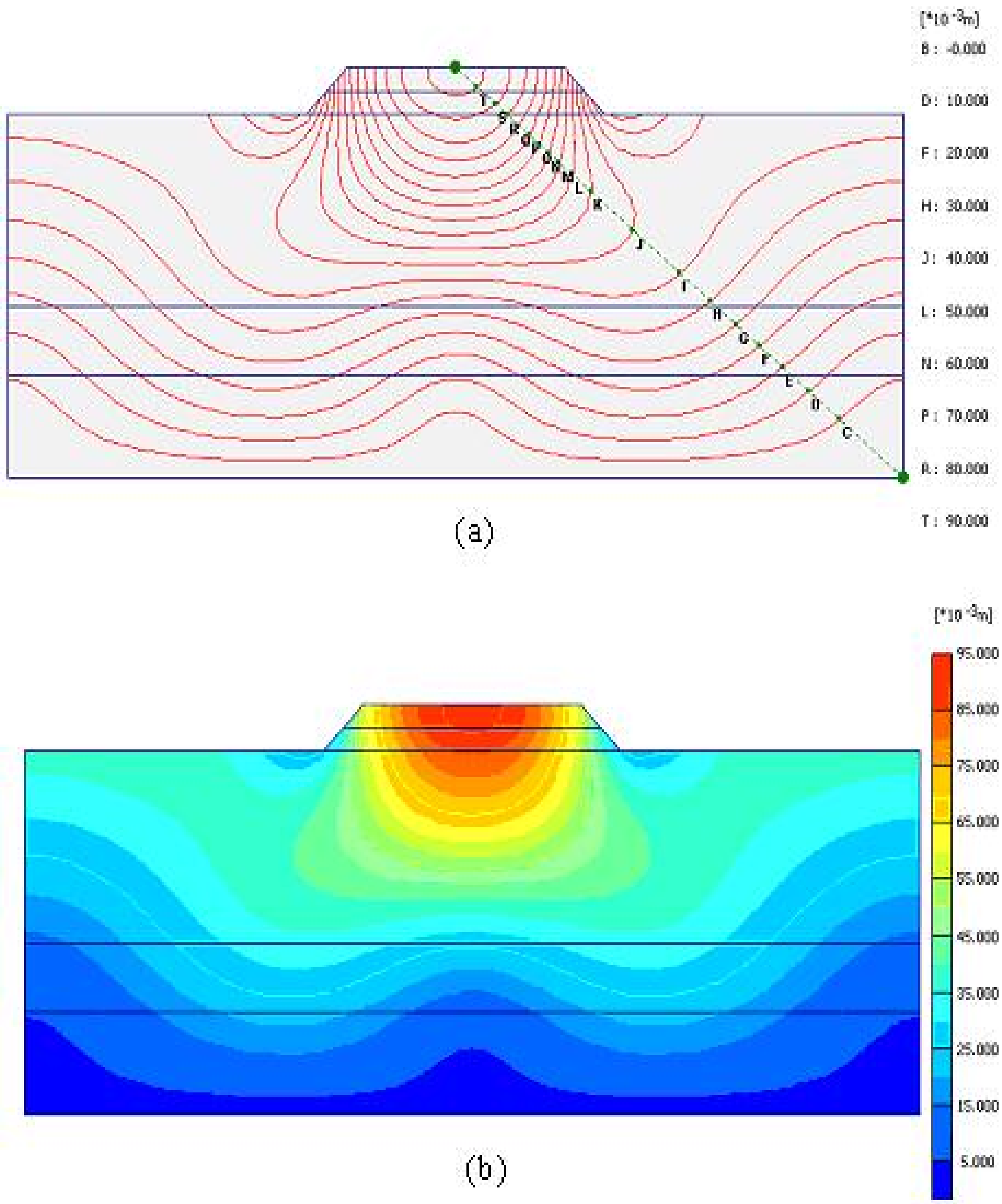


Figure 4.13 FEM Plot of Extreme Total Displacement for 2m Lift of Embankment (Untreated) of 91.78*10^{-3}m (a) Contour Lines (b) Shading

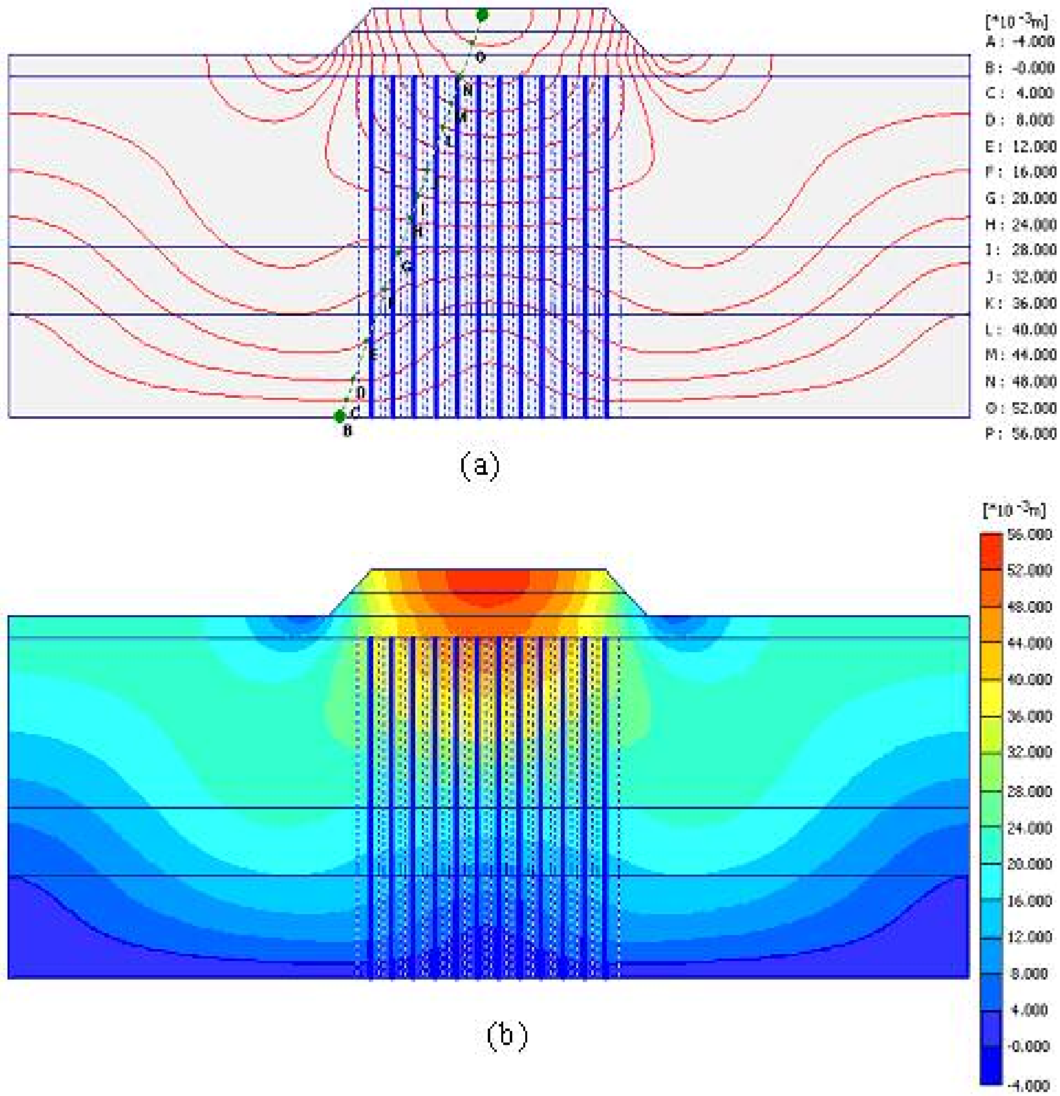


Figure 4.14 FEM Plot of Extreme Total Displacement for 2m Lift of Embankment (Treated) of $54.22*10^{-3}$m (a) Contour Lines (b) Shading

4.7.8 Consolidation Analysis

Automatic time stepping procedure is considered in consolidation analysis. This procedure of automatic time stepping procedure will choose appropriate time steps for a consolidation analysis. A consolidation analysis is performed for 42 days and the deformation outputs are presented in Figure 4.15 and Figure 4.16 for untreated and treated ground respectively. In the FEM analysis, when the calculation runs smoothly, resulting in few iterations per step then the program automatically choose a larger time step. However, when the calculation uses many iterations due to increase in the plasticity, then the program will take smaller time steps.

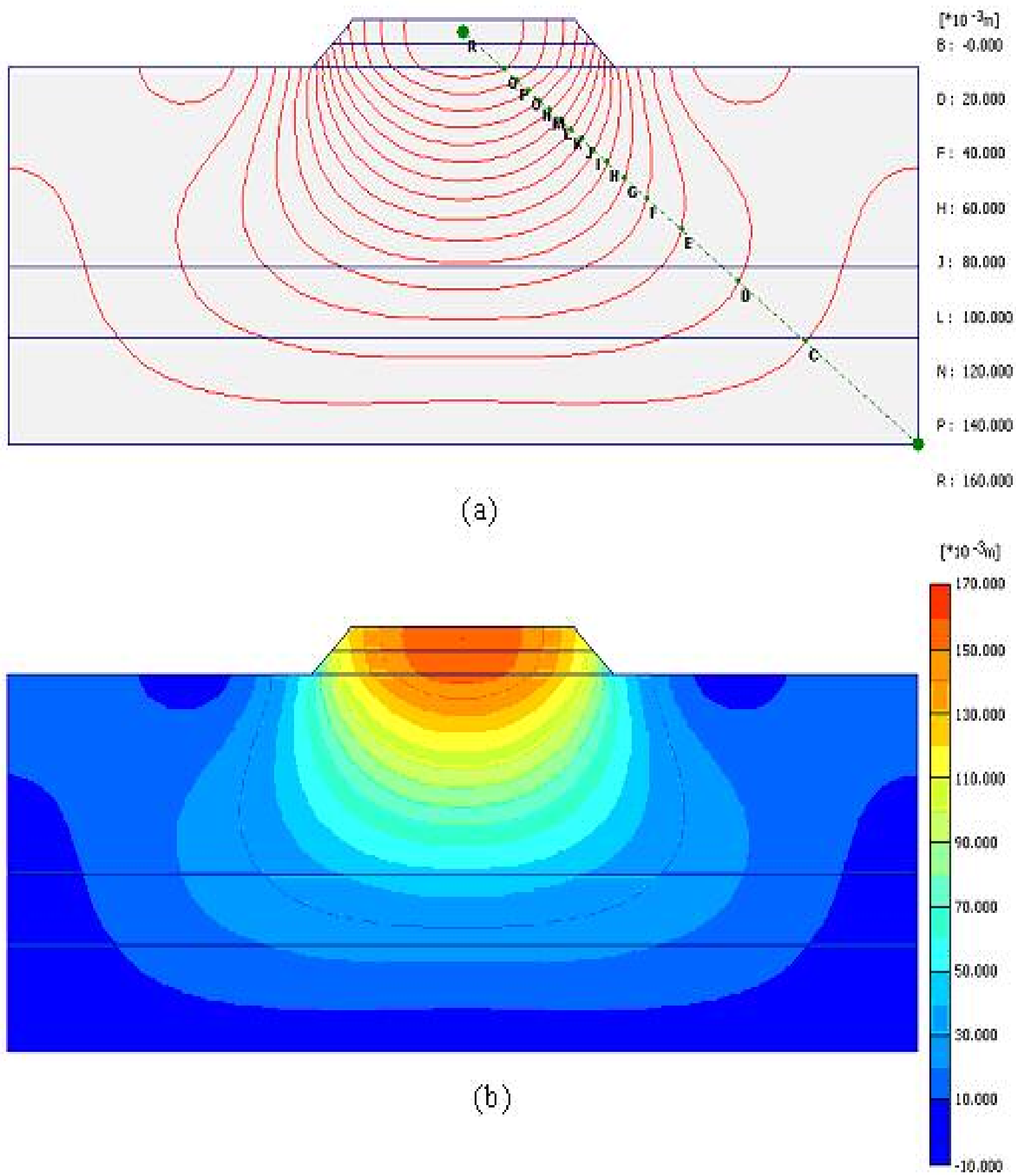


Figure 4.15 FEM Plot of Extreme Total Displacement for Consolidation Analysis (Untreated) $160.05*10^{-3}$m (a) Contour Lines (b) Shading

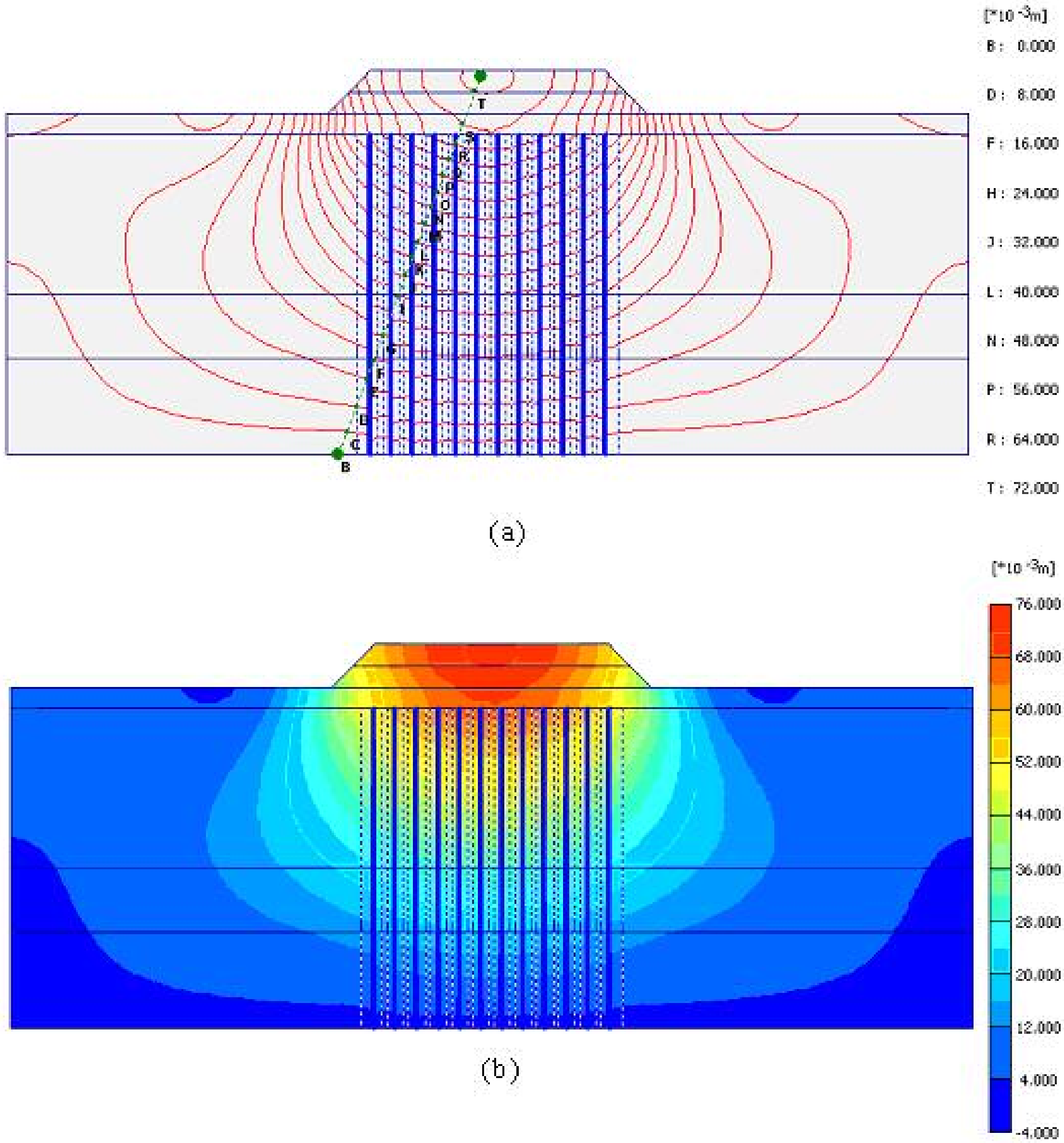


Figure 4.16 FEM Plot of Extreme Total Displacement for Consolidation Analysis (Treated) $72.69*10^{-3}$m (a) Contour Lines (b) Shading

4.7.9 Minimum Pore Pressure Analysis

An extra criterion for terminating the consolidation analysis had been adopted in the present section of model analysis. The numbers of additional steps will be not be reached if prescribed excess pore pressure (*P-Stop)* criterion prevails. The analysis calculation or the iteration will stop if the maximum absolute excess pore pressure is below the prescribed value of *P-Stop.* The deformation outputs of this stage are presented in Figure 4.17 and Figure 4.18 for untreated and treated respectively.

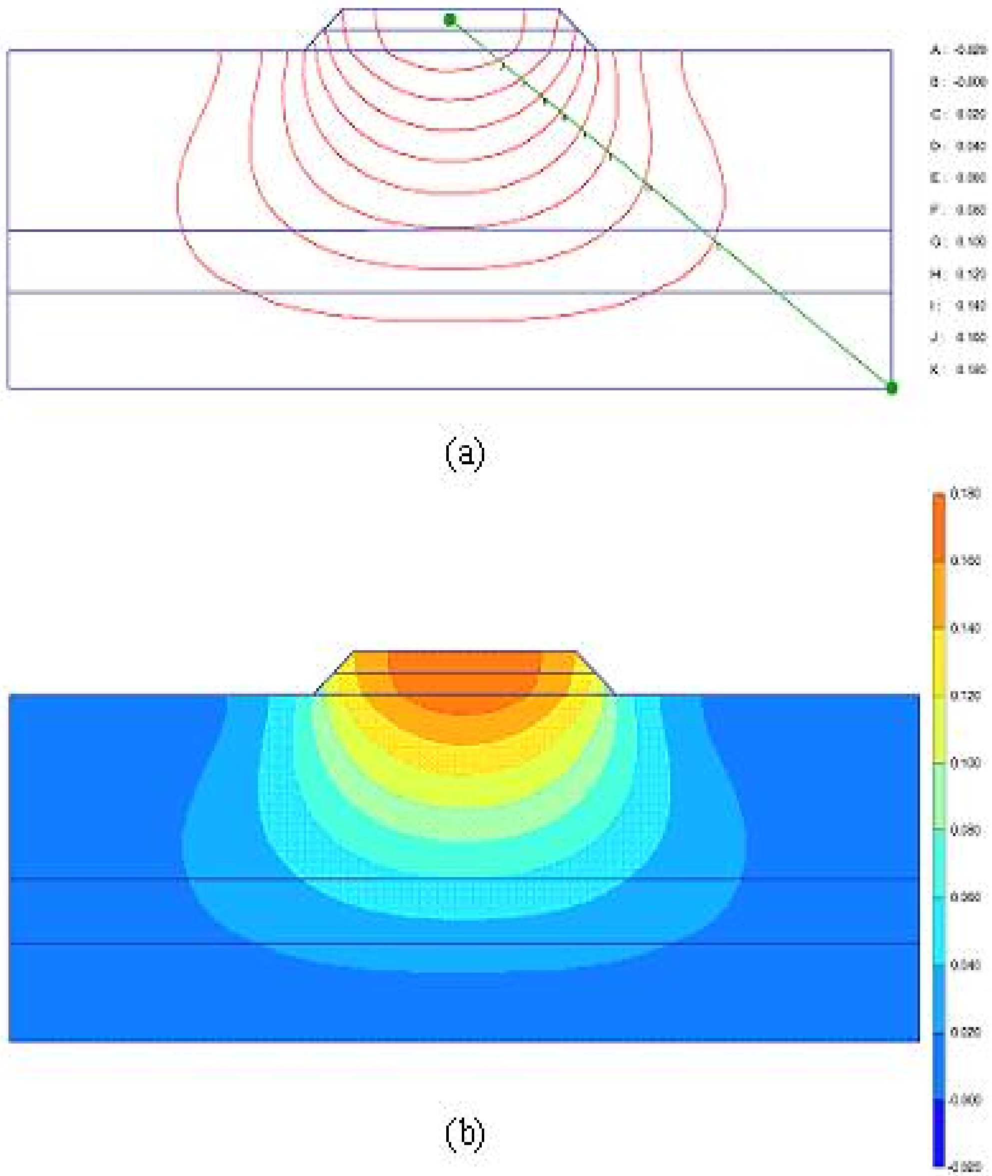


Figure 4.17 FEM Plot of Extreme Total Displacement for P-Stop Analysis (Untreated) $177.08*10^{-3}$m (a) Contour Lines (b) Shading

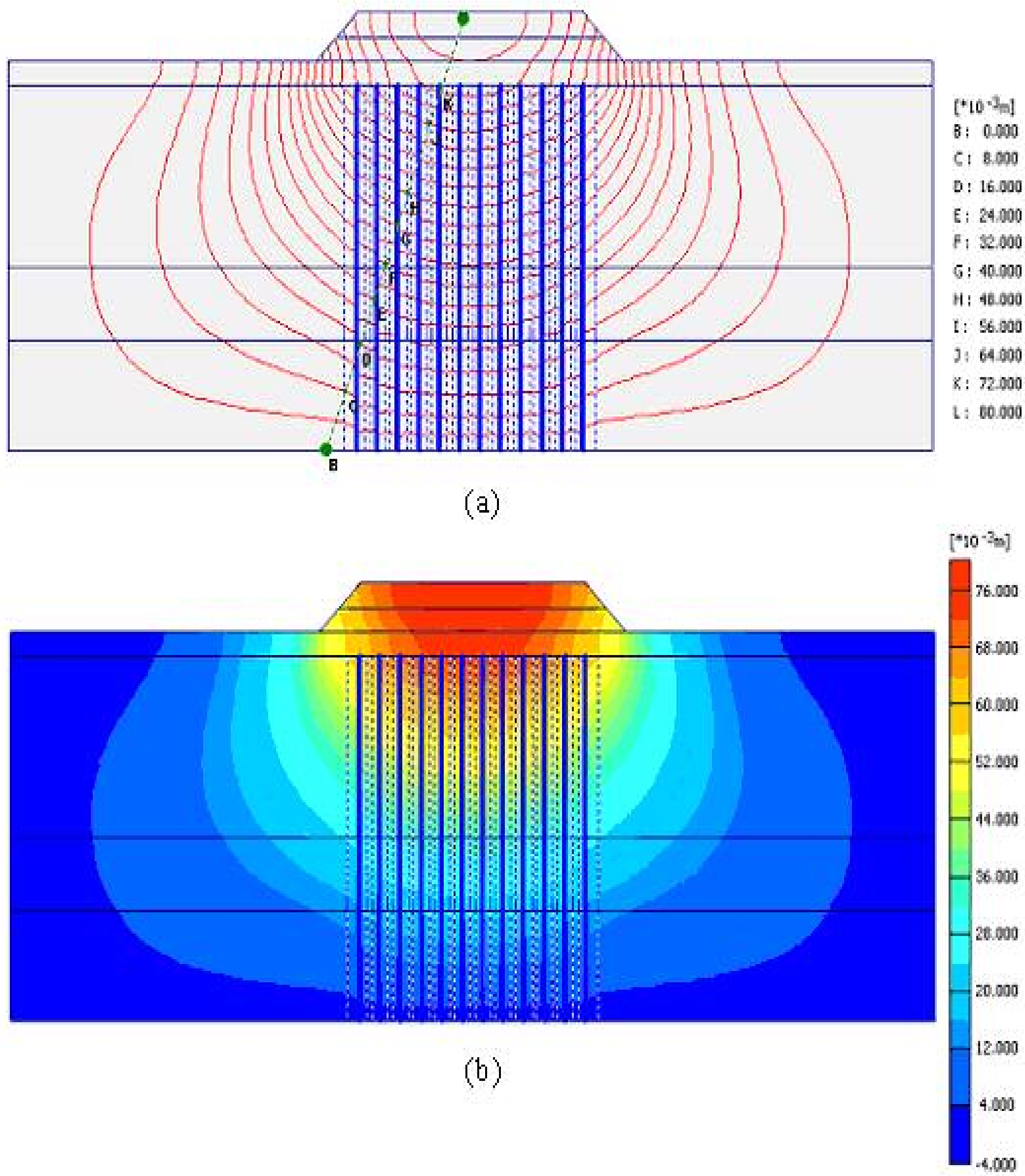


Figure 4.18 FEM Plot of Extreme Total Displacement for P-Stop Analysis (Treated) $79.36*10^{-3}$m (a) Contour Lines (b) Shading

4.8 Comparison between Field Data and Finite Element Results

The predicted embankment behavior by using finite element program PLAXIS has been compared with actual field data in terms of excess pore pressure, settlements, and lateral displacement. The embankment model and generated mesh for the numerical modeling is presented in Figure 4.5. All field data are collected from the Onoda Cement Company Report (1996). The typical variations of excess pore water pressure with time at different depths (3m, 7m, and 14m below the ground surface) have been presented in Figure 4.19 and compared with the FEM results. Initially, the predicted excess pore water pressures are higher than the field data. However, after 15 days the predicted excess pore water pressures are under-predicated. This is mainly due to the difference in simulation of embankment loading in PLAXIS and actual embankment loading.

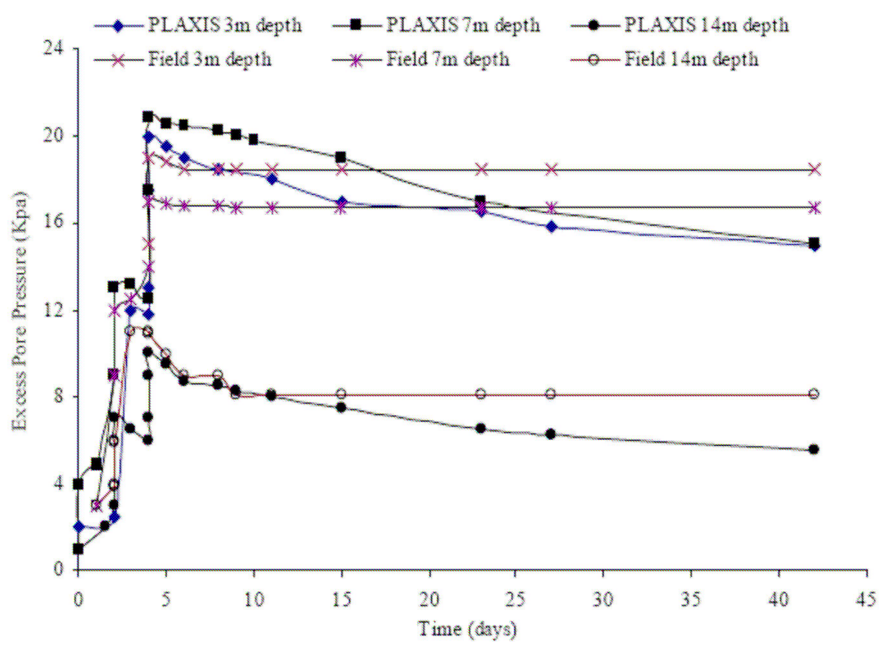


Figure 4.19 Comparison of Predicted and Measured Excess Pore Water Pressure

In the field embankment loading of 2m is completed uniformly over 4 days. However, in FEM simulation the loading is considered as two steps: (i) First 1m of embankment loading is placed instantaneously after 2 days and second 1 m of embankment loading is completed instantaneously after 4 days. Hence, initially, the predicted pore water pressure higher than the actual field data. In addition, in FEM analysis, initially the plastic calculation is performed followed by the consolidation analysis. During the plastic analysis, it is assumed that the load has reached its ultimate state and then consolidation takes places. Once, the load reaches its ultimate state the developed excess pore water pressure during the loading starts dissipating, causing under prediction of pore water pressure at later stages. The variations of predicted and observed field data are not significantly different and the result matches quite well. The predicted and measured surface settlement under the center point of the embankment is presented in Figure 4.20.

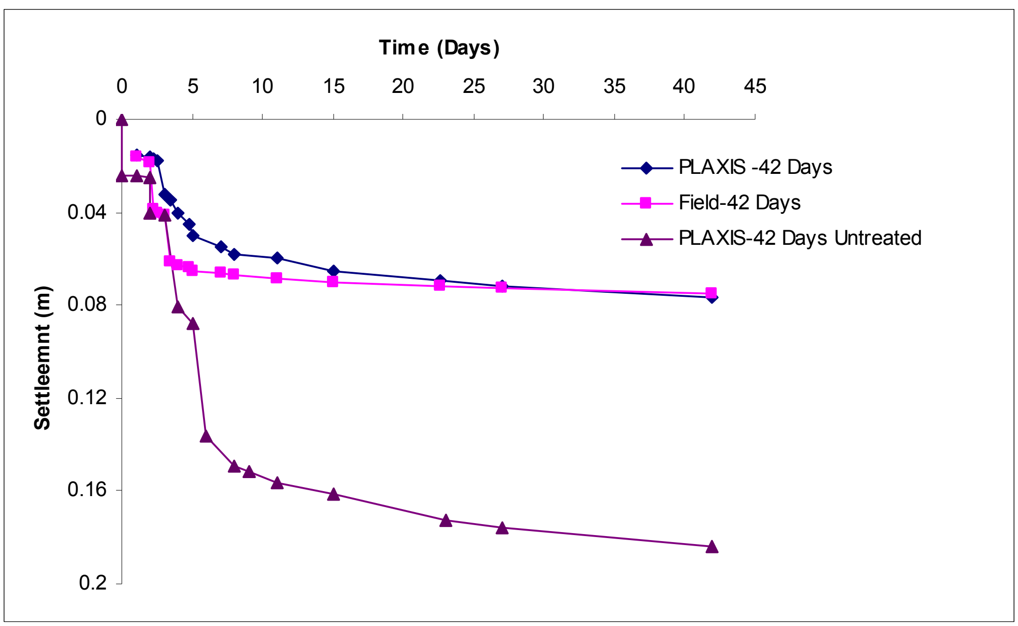


Figure 4.20 Comparison of Predicted and Measured Surface Settlement

Initially, the finite element over-predicts settlement due to difference in simulation of embankment loading for FEM and actual embankment loading. The FEM analysis assumes that the 1st 1m stage loading did not take any time and was placed instantaneously.

Therefore, immediate settlement is very high compared to measured field settlement. The second 1m of stage loading after 4 days was also assumed to be taken place instantaneously and the settlement continues to be higher than the measured value. The measured and predicted settlement is almost same after 25 days. The predicted and measured sub-surface settlement at different depths of the embankment are compared and presented in Figure 4.21. In both the cases, the predicted settlement is more than the filed settlement value. The FEM over predicts the settlement for the same reason as discussed in previous section.

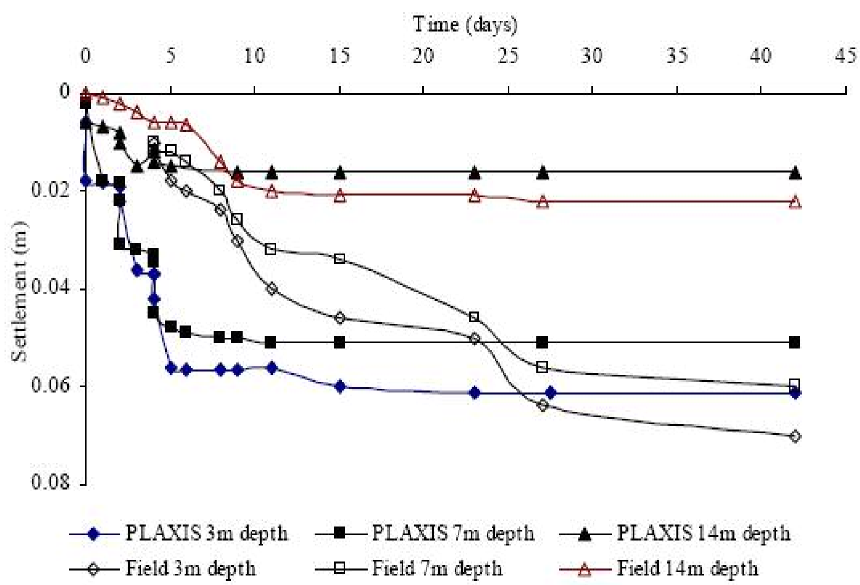


Figure 4.21 Comparison of Predicted and Measured Subsurface Settlement

In addition, during the plastic calculation PLAXIS assumes that the soil has reached its ultimate state, which may not be the situation in field. Because in the field, it takes long time to reach the ultimate plastic state. The settlement profile (Contour and

Shading) obtained from numerical modeling of treated ground is presented in Figure 4.18. The settlement profile shows that the settlement is maximum at the middle and at ground surface. The settlement keeps decreasing with the increasing depth. In addition, the settlement is more at the center than the toe of the embankment, which is actually the case in the field. Figure 4.22 compared the finite element results and measured lateral displacement profile at the inclinometer position.

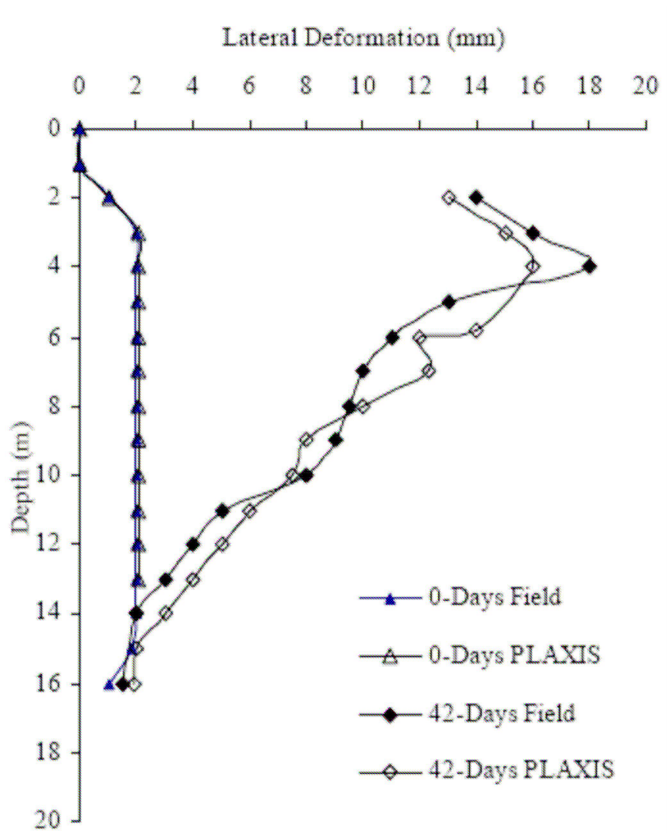


Figure 4.22 Comparison of Predicted and Measured Lateral Deformation

It shows that the agreement between the calculated and measured data is good at the beginning as well as after 42 days. However, after 42 days in the upper soil layer (4 m to 6 m) finite element over-predicts slightly but it is within very close range.

4.9 Summary

The Chemico-pile soil improvement method has been used in a full-scale embankment project located at Nong Ngo Hao site near Bangkok. Both laboratory and field investigations were performed to evaluate the effectiveness of lime column on soft soil. Instrumented embankment behavior was observed for 42 days after embankment load was increased to 2m height. Utilizing the improved soils parameters, embankment behavior on improved ground was predicted using the Finite Element Program PLAXIS. Finally, the field embankment behavior was compared with the predicted FEM results, and the following summary and conclusions are presented:

- The natural water content of improved soils decreased about 20-30% and tends to be constant after 30 days. The reduction is mainly due to the hydration effect and pozzolanic reaction of quick lime with clay.
- Significant changes in void ratio were observed due to dewatering effect of the clayey soil from hydration of quick lime. The compressibility ratio also decreased up to 55% for the very soft clay.
- The undrained shear strength of improved soil decreased with time for 30 days of curing period and then the strength increased. Initial reduction in strength was due to the disturbance of soft clay during installations of the Chemico-pile. The shear strength increased by 50-70% after 90 days.
- Initially, the predicted excess pore water pressure of improved ground is higher than the actual field data and after 15 days, the predicted results are similar to that of actual field data. The main reason for the initial over prediction was the

modeling of embankment loading. In the final element analyses, first 1m and 2nd 1m load was applied instantaneously after 2 days and 4 days, respectively. However, in the field the 4m load was applied uniformly over four days. The reason for under-prediction after 15 days in PLAXIS is that first, the plastic calculation is done and then consolidation analyses. In plastic calculation, first the load has to reach the ultimate value and then the consolidation calculation. In the field, it really takes long time to reach ultimate value and therefore, the excess pore water pressure is a bit high.

• The settlement behavior is same as the pore water pressure for the same reason as explained in the previous section. In addition, the two steps curves are due to loading in two stages.

• The measured lateral deformation is very similar to the predicted value form PLAXIS analysis. The comparison indicates the Chemico-pile improved embankment behavior can be predicted quite accurately with the selection of proper soil parameters. The prediction results matched quite well with the actual field results.

CHAPTER 5

PARAMETRIC STUDIES AND RESULTS

5.1 Introduction

The application of pile supported embankment affirms to be an effective alternative solution to conventional solutions for the problems confronted on highly compressible soils. Installation of geosynthetic reinforcement increases the load transfer and reduces the area replacement ratio of the columns (piles). It is observed that the stress concentration ratio increases with increase in tensile stiffness of geosynthetic reinforcement and elastic modulus of pile. The load transfer from soft soil to piles is mainly due to soil arching. Geosynthetic reinforcement enhances the load transfer from the fill soil to piles, reduces the total and differential settlements. Further reducing the differential settlements at the base of the embankment, this is reflected at the surface of the embankment. Increase in shear strength leads to increase in load transfer. Various methods have been adopted for design and analysis of pile supported embankments. A series of model test have been conducted in order to analyze the influence of shear strength on soft soils. The influence of various geosynthetics stiffness, pile types, and soil modulus on the effectiveness of pile supported embankment can be inferred with the reduction in deformation in the model. Installation of geosynthetic reinforcement increases the load transfer and reduces the area replacement ratio of the columns (piles). The stress concentration ratio increases with increase in tensile stiffness of geosynthetic.

5.2 Analysis and Results

5.2.1 Effects of Piles

The embankment analysis with different elastic modulus, which represents different pile materials for the existing ground conditions, has been considered. The different pile materials consider for the analyses are (a) Stone Column (b) Deep Mix Column (c) Treated Timber and (d) Concrete Piles. Stage construction of the embankment with a lift of one meter per stage has been analyzed. The embankment model properties and the Axial Stiffness (EA) and Flexural Rigidity (EI) of pile considered in the analyses are tabulated in Table 5.1 and Table 5.2.

Table 5.1 Soil Data Set Parameters for FEM Analysis

Mohr-Coulomb		Chemicolizer	Very soft Clay	Soft Clay	Medium Clay	Embankment
Type		Undrained	Undrained	Undrained	Undrained	Drained
γ_{unsat}	[kN/m³]	15.00	16.00	17.00	18.00	20.00
γ_{sat}	[kN/m³]	22.00	16.00	17.00	18.00	22.00
k_x	[m/day]	0.009	0.005	0.001	0.001	1.000
k_y	[m/day]	0.001	0.005	0.001	0.001	1.000
E_{ref}	[kN/m²]	20000.000	2100.000	2300.000	2900.000	8000.000
υ	[-]	0.300	0.300	0.300	0.300	0.300
c_{ref}	[kN/m²]	200.00	21.00	23.00	29.00	1.00
φ	[°]	0.00	0.00	0.00	0.00	30.00
Ψ	[°]	0.00	0.00	0.00	0.00	0.00
$R_{inter.}$	[-]	1.00	1.00	1.00	1.00	1.00

Table 5.2 Pile Stiffness Properties

Pile Type	Axial Stiffness (EA- kN/m)	Flexural Rigidity (EI- kNm^2/m)
Stone Column (Datye, 1982)	2513.274	26.32
Deep Mix Column (Kitsugi & Azakami, 1985)	33087.2522	346.5028
Treated Timber (FLAC Manual)	329176.0622	3447.262
Concrete pile (Reinaldo & Yong, 2003)	987088.36	10337.18

The Equivalent Young's Modulus (E_{eq}) for combination of columns and soil need to be computed when deep mix column (instead of beam element for columns) are considered for the embankment analysis by soil replacement technique in PLAXIS. The expression for the E_{eq} is given as;

$$E_{eq} = E_p\, a_s + E_s\,(1 - a_s) \tag{5.1}$$

Where, E_p : Young's Modulus for Pile

E_s : Young's Modulus for soil

a_s : Ratio of Treated Area to Total Area

A surface ground improvement is also adopted in the analysis. The chemicolizer, which is one of the ground improvement techniques, has been adopted to contrast the embankment response without any kind of stabilization (or treatment). In chemicolizer method, a lime-based compound is mixed with the top layer of the existing ground to increase the shear strength. The depth up to which the stabilization is done depends on the existing ground conditions. The stabilized soil should be well compacted to achieve the required improvement. The dehydration due to lime mixing leads to lime slacking. Evaporation of water is observed due to generated heat. The reaction is as mentioned below;

$$CaO + H_2O \text{ ------> } Ca(OH)_2 + 1.56 \text{ Kcal/mol} \qquad (5.1)$$

The water content for the improved soil can be expressed as follows;

$$w' = \frac{(w_n - 77a_w)}{1 + 1.32a_w} \% \qquad (5.2.)$$

Where, a_w : Ratio of Lime weight to Dry weight of soil

w_n : Initial water content (%)

w' : water content of soil after lime addition (%)

The effects of pile system (Stone Column, Deep Mix Column, Treated Timber and Concrete column) are analyzed with and without surface treatment (Chemicolizer). The existing ground conditions with 2m lift of fill embankment is considered in the present analysis. The parameters in the current are summarized in Table 5.3.

Table 5.3 Effects of Pile System Analysis

Case	Pile Type	Surface Treatment	Conditions
A	Stone Column	With Chemicolizer & Without Chemicolizer	Existing Ground Conditions and 2m High Embankment
B	Deep Mix Column		
C	Treated Timber		
D	Concrete pile		

The deformation analyses for different piles with and without shallow stabilization are illustrated graphically. The FEM analysis for the Pile system with and without the surface treatment is illustrated in Figure 5.1. It can be inferred from the

FEM outputs that there is 60% improvement in stabilization from stone column to concrete column for the model without surface treatment as compared to 83% improvement in stabilization with the surface treatment. In addition to the significance of surface treatment, it is also evident that the model response for the treated timber column and concrete column is similar.

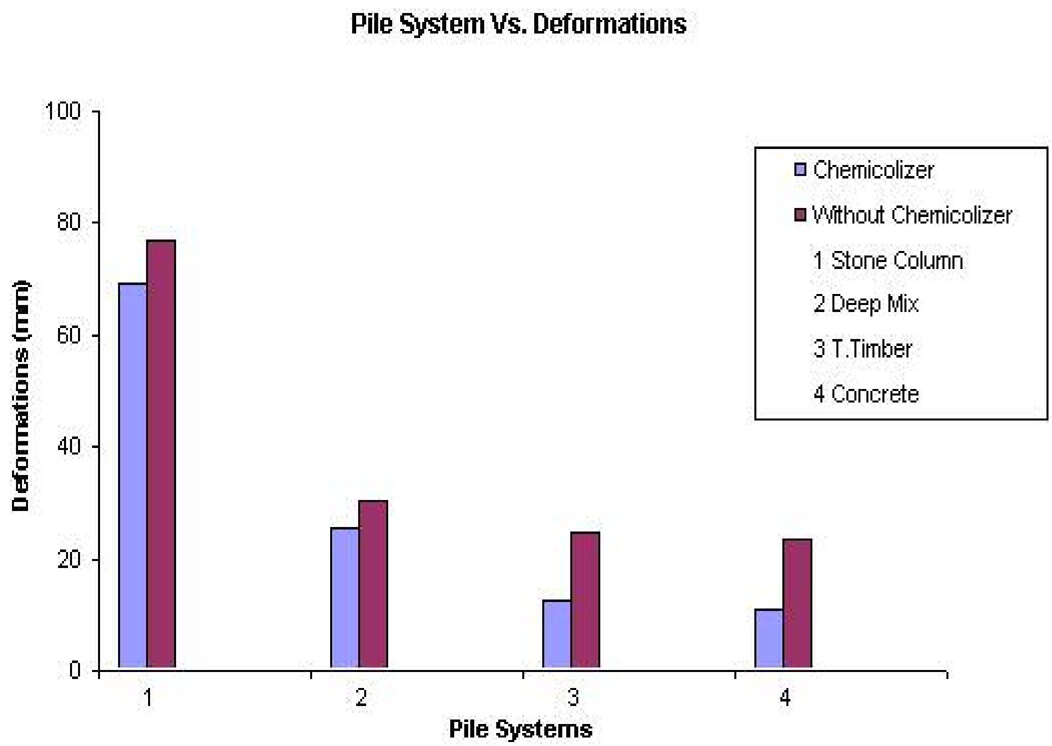
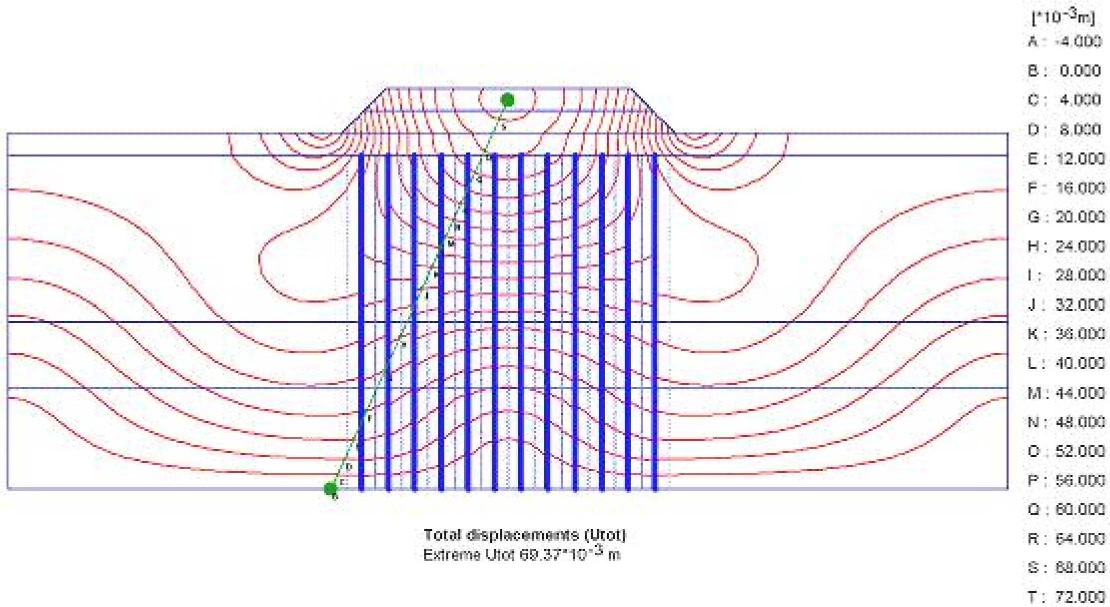
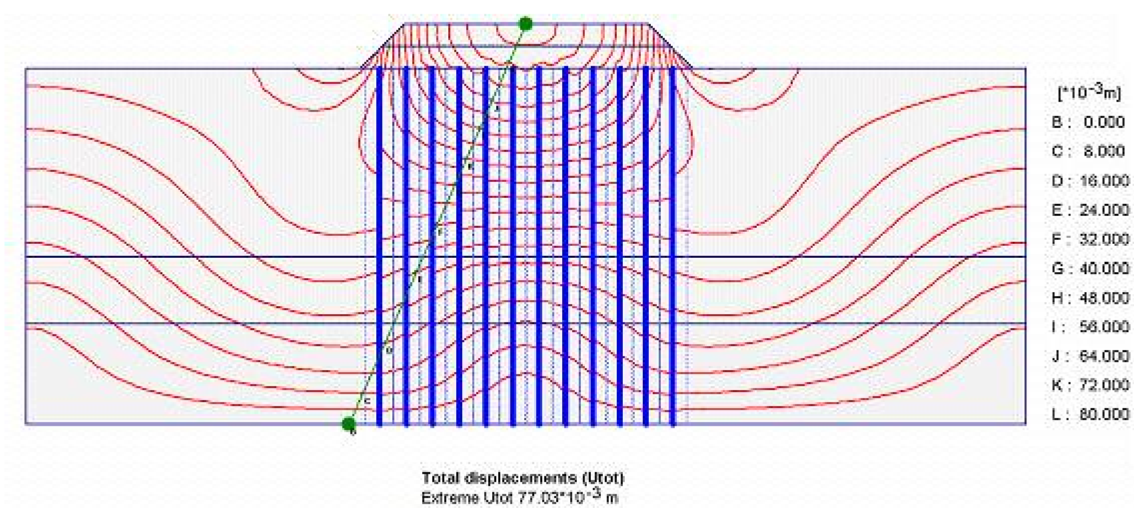


Figure 5.1 Deformation Analyses for Pile Systems With and Without Surface Treatment

The FEM deformation analyses for stone column, deep mix column, treated timber and concrete column with and without shallow stabilization are illustrated in Figure 5.2, Figure 5.3, Figure 5.4, and Figure 5.5 respectively. The contour plots are illustrated for different pile systems with and without surface treatment. It can be observed from the FEM contour plots that there is a variation in the soft ground disturbance due to the presence of low stiffness stone column. A maximum deformation improvement of 87% is achieved in the current analysis, which is evident in the deformation plots from the finite element analysis.

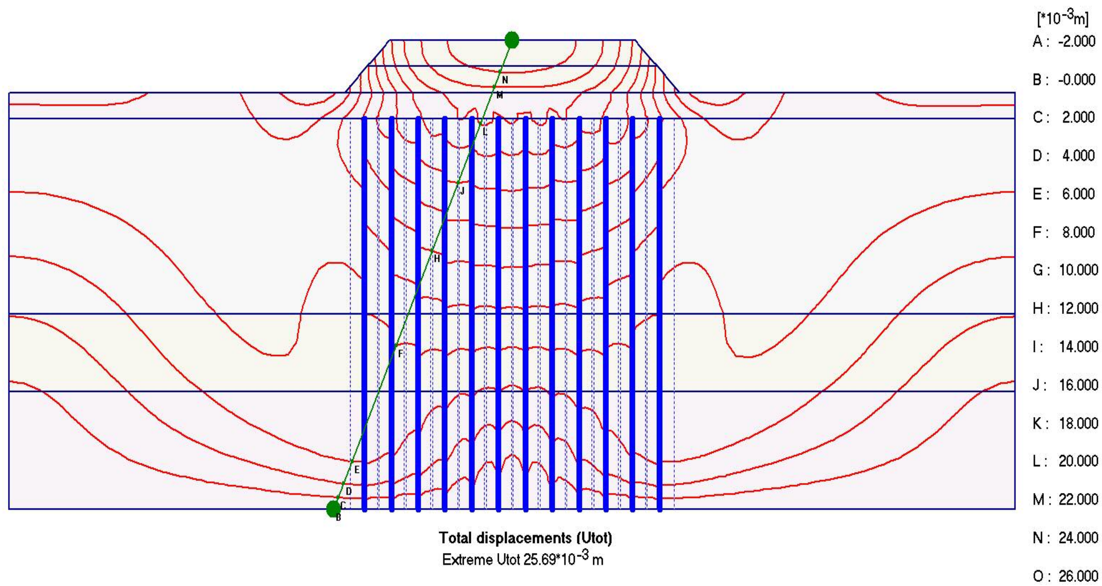


(a)

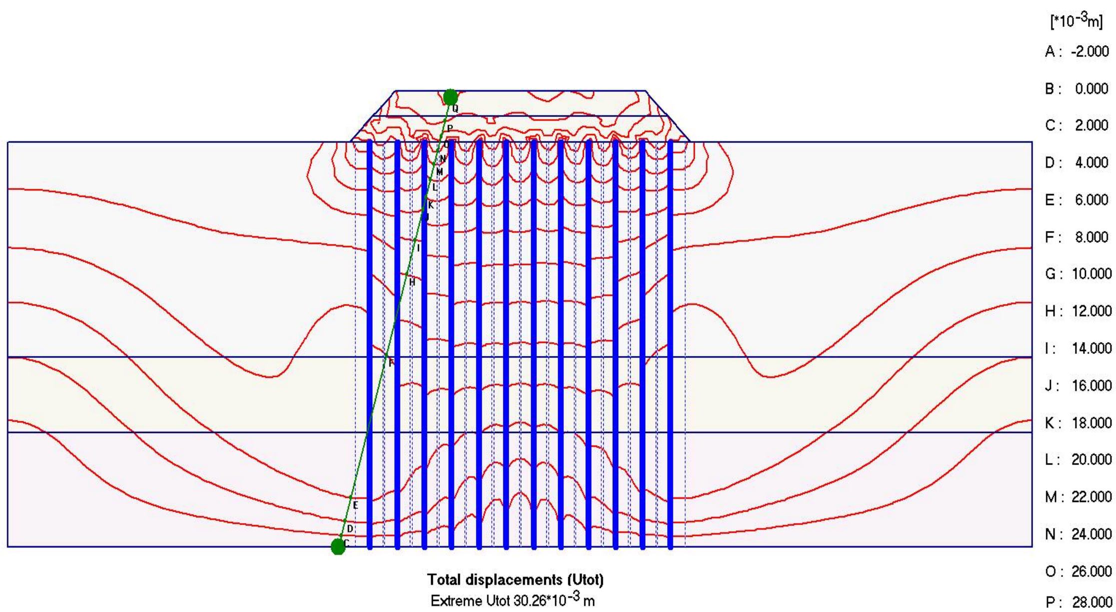


(b)

Figure 5.2 Deformation Analyses for Stone Column (a) With Surface Treatment ($69.37 * 10^{-3}$ m) (b) Without Surface Treatment ($77.03 * 10^{-3}$ m)

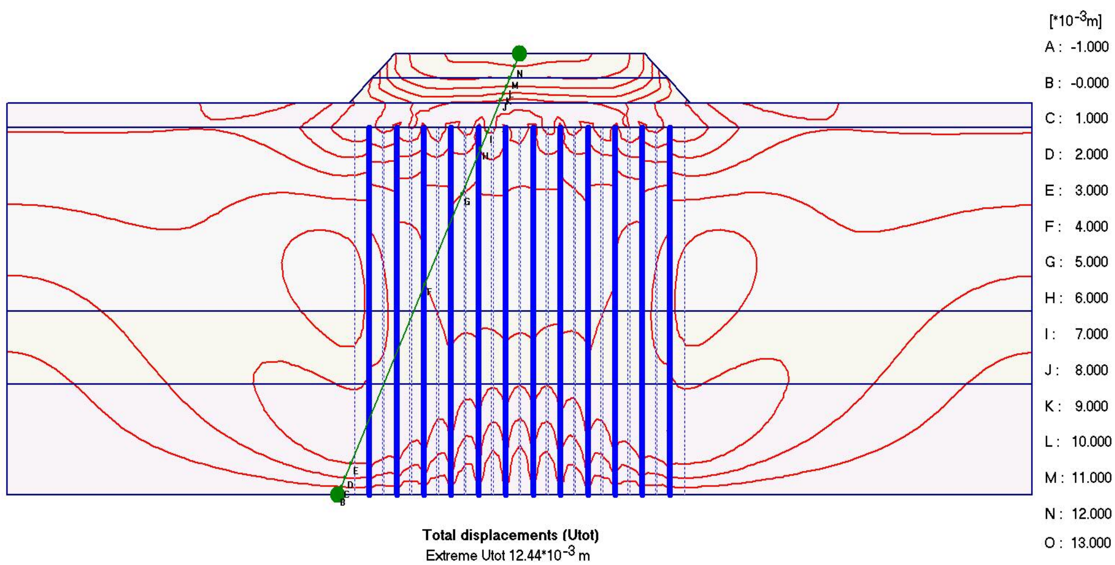


(a)

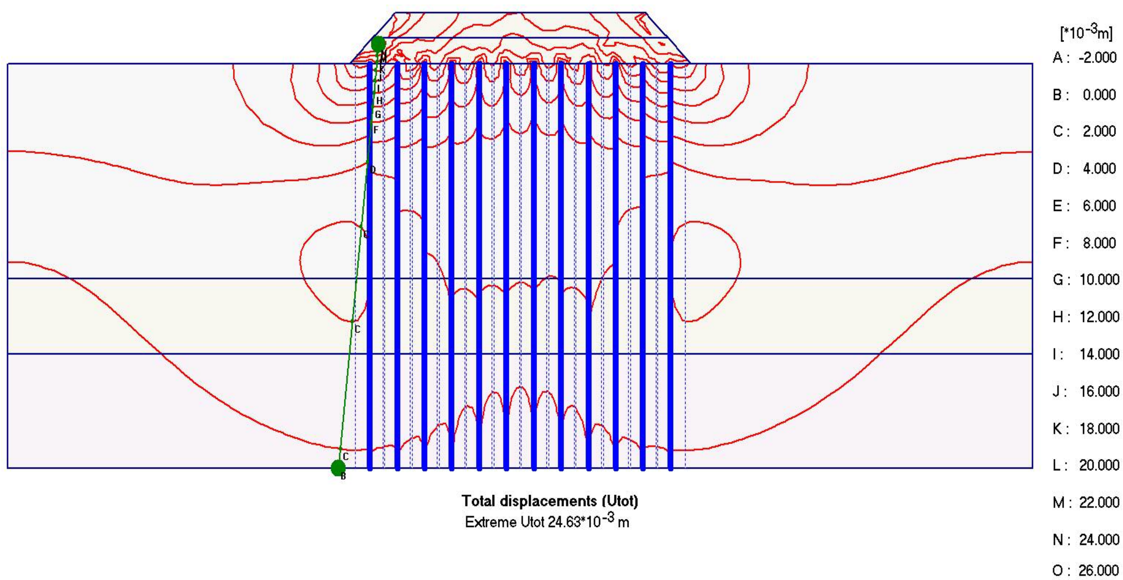


(b)

Figure 5.3 Deformation Analyses for Deep Mix Column (a) With Surface Treatment ($25.69 * 10^{-3}$ m) (b) Without Surface Treatment ($32.26 * 10^{-3}$ m)

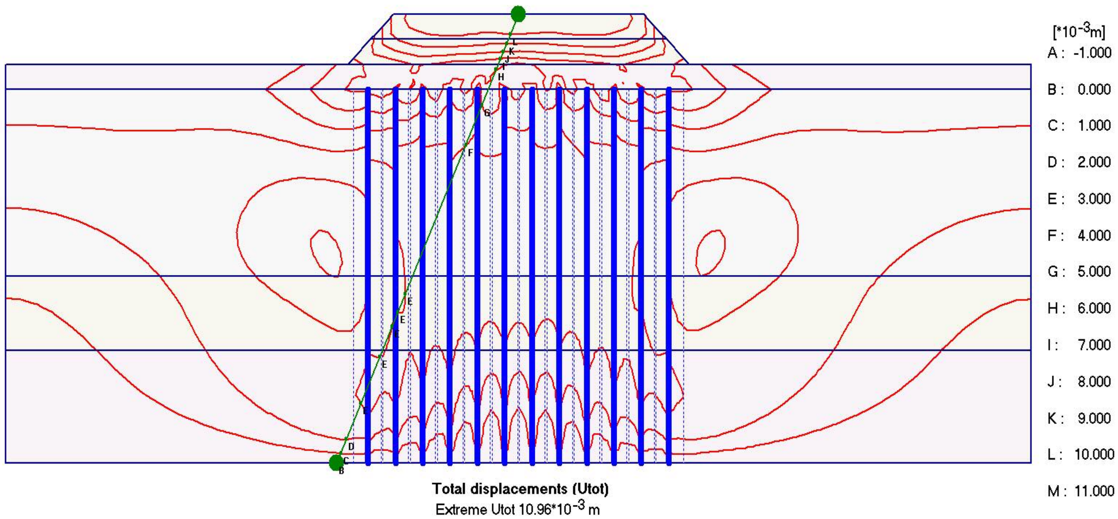


(a)

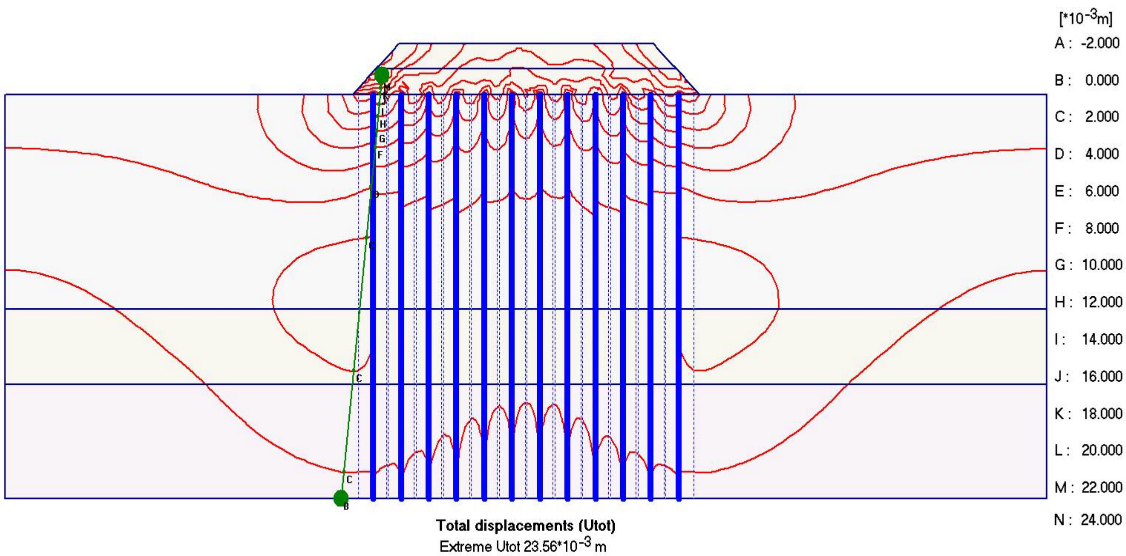


(b)

Figure 5.4 Deformation Analyses for Treated Timber Column (a) With Surface Treatment (12.44* 10^{-3} m) (b) Without Surface Treatment (24.63* 10^{-3} m)



(a)



(b)

Figure 5.5 Deformation Analyses for Concrete Column (a) With Surface Treatment ($11.39* 10^{-3}$ m) (b) Without Surface Treatment ($23.89* 10^{-3}$ m)

5.2.2 Effects of Piles with Geosynthetics

The embankment response to different range of geosynthetics stiffness has been analyzed in combination with different pile materials. The use of geosynthetic reinforcement in pile-supported embankments constructed on soft soils is used to control both initial stability and settlement. Difference in compressibility between the soft soil and incompressible piles leads to complex foundation interaction problem. This leads to soil arching effects in the fill soil. In particular, the differential settlement of pile-supported embankment should be within the allowable limits by optimizing the fill height, pile spacing, and tensile strength of geosynthetics reinforcement and soil stiffness. The principle failure mechanisms according to BS 8006: 1995 for ultimate and serviceability are shown in Figure 5.6 and Figure 5.7 respectively.

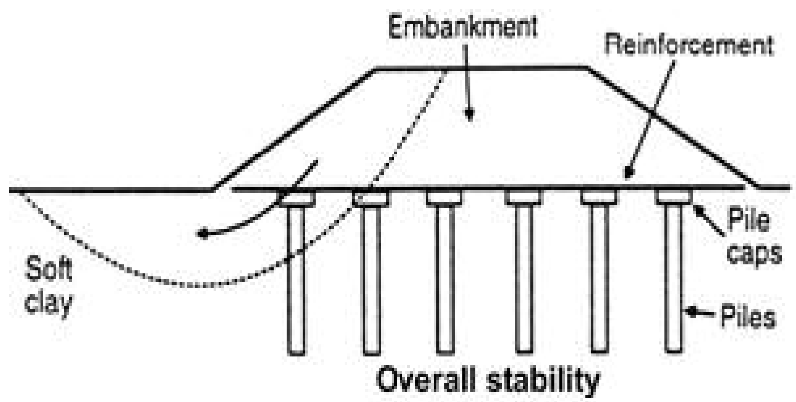


Figure 5.6 Ultimate Limit State for Pile-Supported Embankment
(Floss and Brau, 2003)

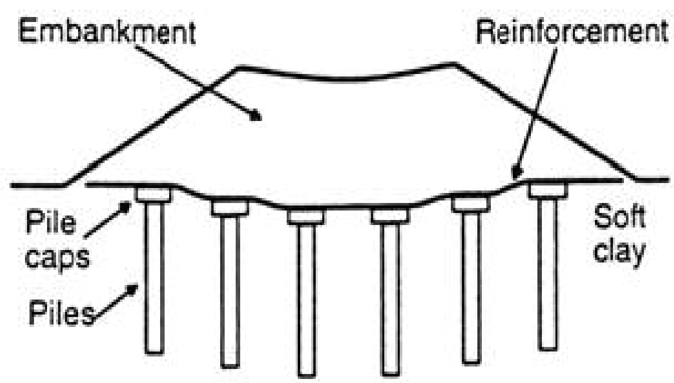


Figure 5.7 Serviceability Limit for Pile-Supported Embankment
(Floss and Brau, 2003)

The soil reinforcement by geosynthetics improves soil by supporting tensile forces. The reinforcing elements with low bending stiffness can transfer axial tensile loads. This improvement reduces the shear force and enhances the shearing resistances in the soil by increasing the normal stress, which is acting on the shear surfaces.

In addition to the wide range of geosynthetic reinforcement stiffness (88 kN/m, 1000 kN/m, 4000 kN/m, 7000 kN/m and 10,000 kN/m) used in the analysis, the location effects of geosynthetic are also considered. The geosynthetic reinforcements are placed at three different embankment elevations (a) At embankment and ground surface interface (b) At 100mm above ground surface in the embankment and (c) At 200mm above ground surface. The geosynthetic reinforcement with varied elevations in combination with different pile materials has been analyzed and the results obtained are illustrated Figure 5.8, Figure 5.9, Figure 5.10, and Figure 5.11. The parameters for the analysis are summarized in Table 5.4.

Table 5.4 Effects of Pile System Analysis

Pile Type	Reinforcement	Conditions
Stone Column	Without Chemicolizer With Geosynthetics Stiffness (a) J1 : 88kN/m (b) J2 : 1000kN/m (c) J3 : 4000kN/m (d) J4 : 7000kN/m (e) J5 : 10,000kN/m	Existing Ground Conditions and 2m High Embankment. Geosynthetics at: (a) Ground Surface (b) 100mm above (c) 200mm above
Deep Mix Column		
Treated Timber		
Concrete pile		

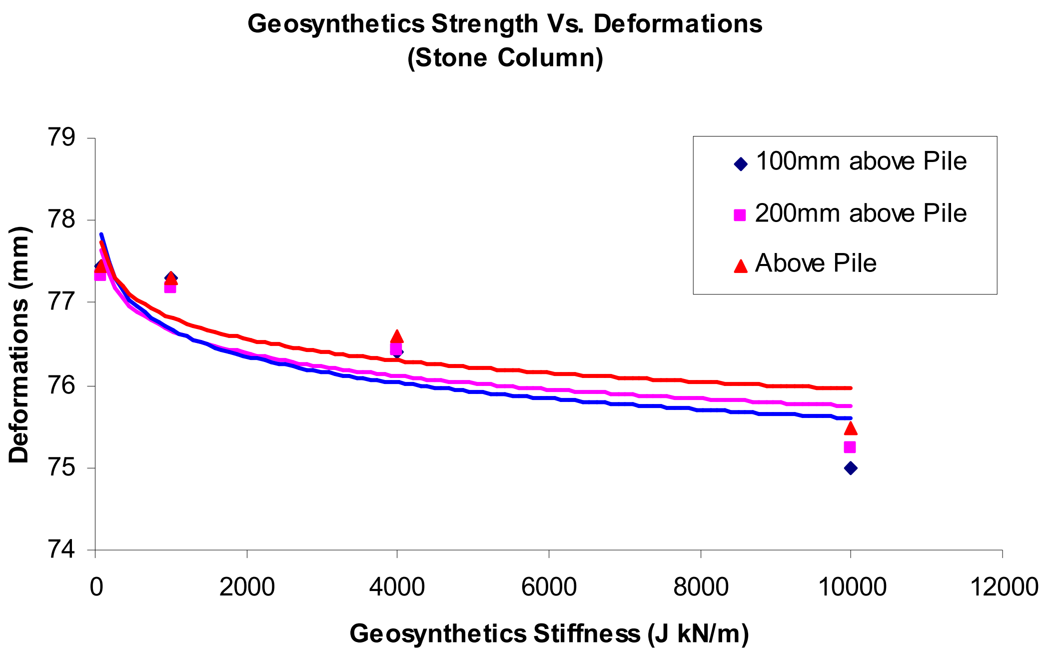


Figure 5.8 Deformation Analyses for Stone Column Supported Embankment

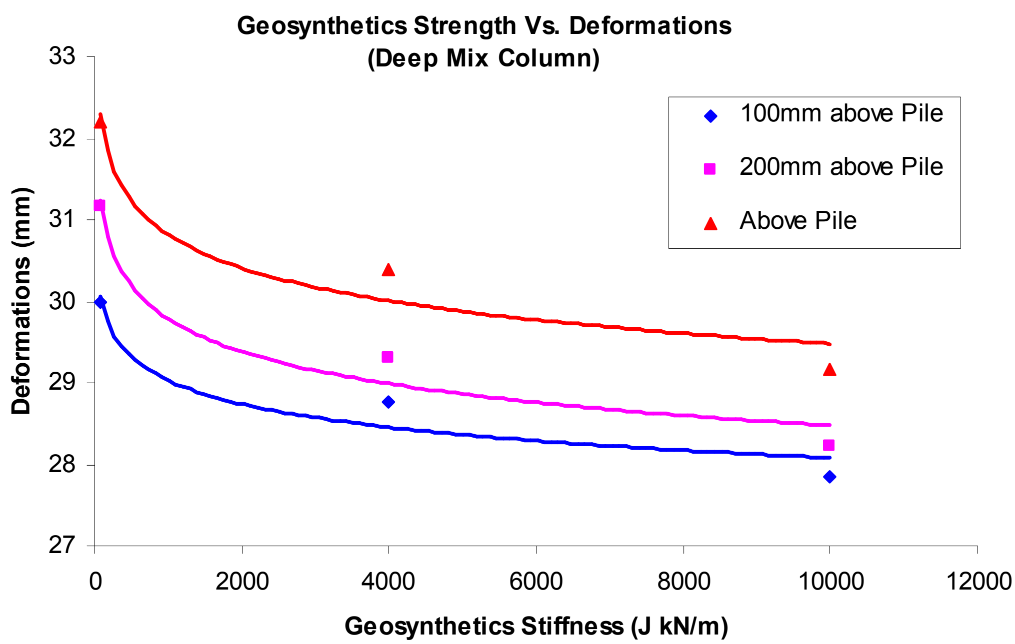


Figure 5.9 Deformation Analyses for Deep Mix Column Supported Embankment

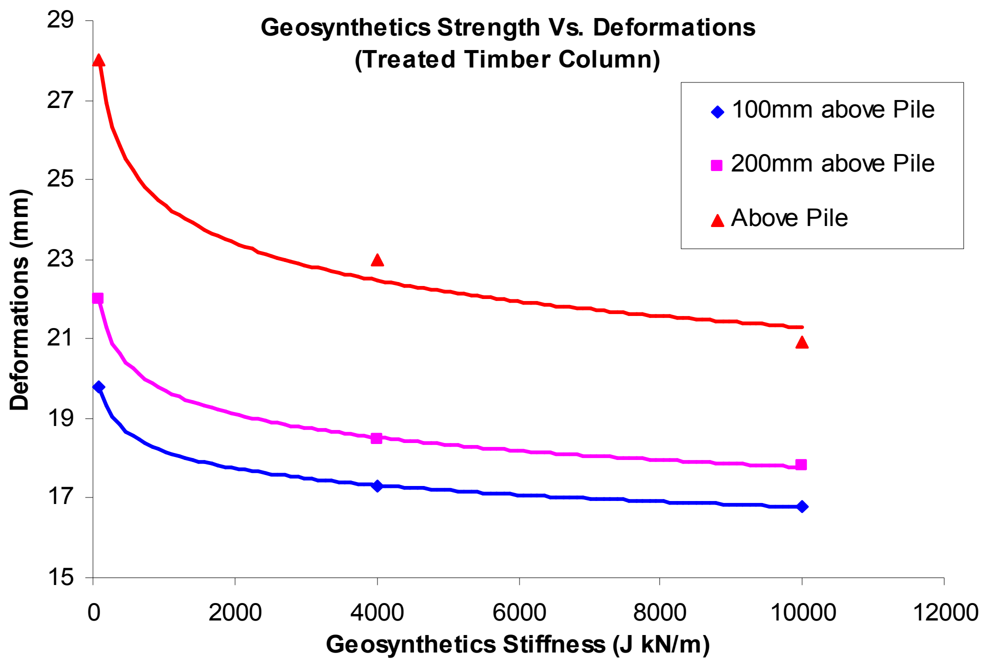


Figure 5.10 Deformation Analyses for Treated Timber Column Supported Embankment

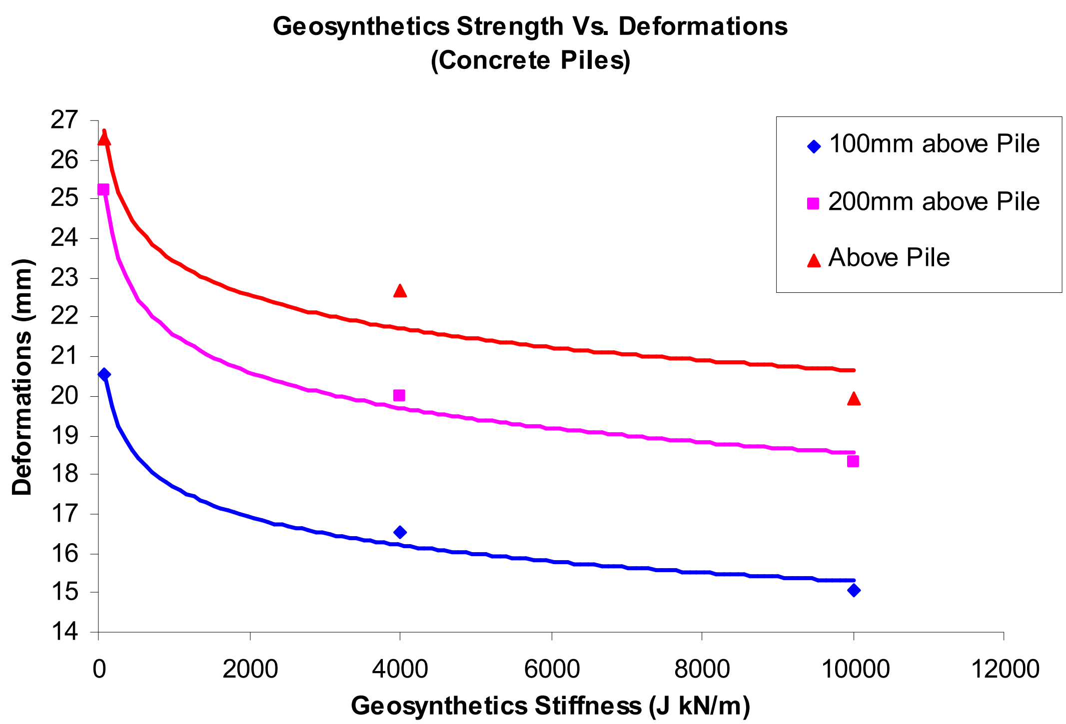


Figure 5.11 Deformation Analyses for Concrete Column Supported Embankment

The effects of geosynthetic stiffness, placement of geosynthetics and variation in pile stiffness are imperative in reducing the model deformation. It can be inferred from the FEM plots that the stone column exhibits maximum deformation as compared to concrete column. It was observed that there is 65% improvement in reduction of model deformation with low stiffness column (stone column) as compared to high stiffness column (concrete). However, it affirms that an additional 20% improvement in reduction of maximum model deformation is observed when the existing model is reinforced with geosynthetics. The elevation, at which the geosynthetic was placed, plays a significant role in reducing the model deformation. The geosynthetic reinforcements are placed at three different embankment elevations (a) At embankment and ground surface interface (b) At 100mm above ground surface in the embankment and (c) At 200mm above ground surface in the embankment. From the FEM plots as illustrated in Figure 5.8, Figure 5.9, Figure 5.10, and Figure 5.11, it can be inferred that the geosynthetic placed at 100mm above ground surface in the embankment is ideal, as there is a considerable reduction in model deformation. In addition to soil arching effects and the stress concentration, the tension member effects of a single layer geosynthetic used in the model, resulted in reduction of model deformation.

5.2.3 Effects of Soil Conditions

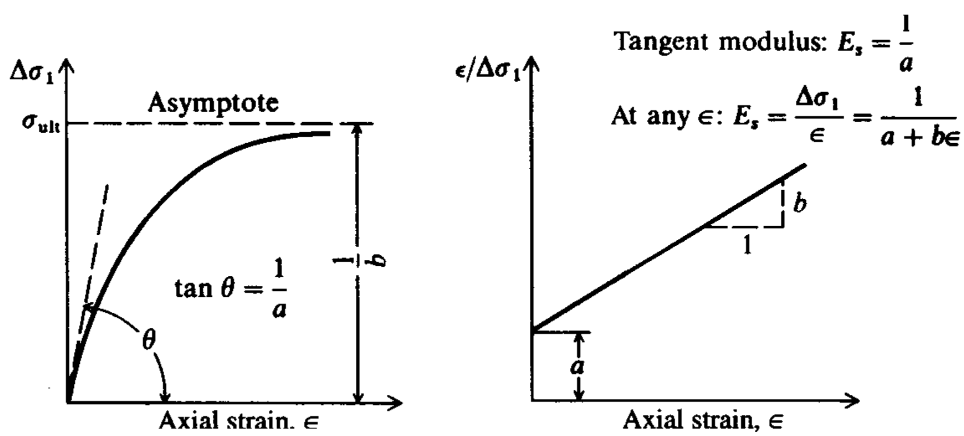
The stress-strain curve for all soil type is non-linear except the initial position of the stress-strain curve. Kondner (1963) has proposed a hyperbolic representation of the stress-strain curve for soils. The theory proposed by Kondner (1963) was in agreement with that of theory proposed by Tan et al., (1991). The usual stress-strain (as in Figure 5.12 a) curve could be represented by the hyperbolic equation:

$$\sigma_1 + \sigma_3 = \frac{\varepsilon}{a + b\varepsilon} \tag{5.3}$$

For $\Delta\sigma_1 = \sigma_1 - \sigma_3$;

$$\frac{\varepsilon}{\Delta\sigma_1} = a + b\varepsilon \tag{5.4}$$

The left side of equation 5.4 can be computed for various values of deviator stress ($\Delta\sigma_1$) and the corresponding strain to make a linear plot as shown in Figure 5.12 b.



(a) (b)

Figure 5.12 Stress-Strain Plots for Soils (a) Hyperbolic Curve Approximation (b) Kondner (1963) Linear Model

The proposed procedure by Kondner for clays can be extended for all kinds of soils with similar stress-strain curves (Joseph, 1996). These arrangements have a particular value in finite element method (FEM) analysis. The computation of E_s is easier. The following empirical correlations are used in estimations of E_s for cohesive soils.

For Normally Consolidated clay

$$E_s = (200 \text{ to } 500)\ S_u \tag{5.5}$$

For Lightly overconsolidated clay

$$E_s = (750 \text{ to } 1200)\ S_u \tag{5.6}$$

For Heavily overconsolidated clay

$$E_s = (1500 \text{ to } 2000)\ S_u \tag{5.7}$$

Considering the lower limit of equation 5.5 suitable for the analysis, a wide range of Young's Modulus have been calculated, which represents modified conditions for exiting ground. The computed moduli for the existing ground and the parameters considered to study effects of soil conditions in the present analysis are tabulated in Table 5.5 and Table 5.6, respectively.

Table 5.5 Young's Modulus for Modified Ground

S_u (kN/m^2)	E (kN/m^2) ($E = 200S_u$)
1	200
2.5	500
5	1000
15	3000
50	10000

PLAXIS analysis to study the effects of modified ground conditions in conjunction with varying geosynthetic stiffness (88 kN/m, 1000 kN/m, 4000 kN/m,

Table 5.6 Parameters Considered to Study Effects of Soil Conditions

Geosynthetics Strength	Soil Modulus	Conditions
J1 : 88kN/m	$E = 200S_u$ (a) E1 : 200 kN/m^2 (b) E2 : 500 kN/m^2 (c) E3 : 1000 kN/m^2 (b) E4 : 3000 kN/m^2 (e) E5 : 10000 kN/m^2	2m High Embankment Without Pile System
J2 : 1000kN/m		
J3 : 4000kN/m		
J4 : 7000kN/m		

7000 kN/m) have been illustrated in the following Figures. The influence of geosynthetic location has been analyzed and is presented in Figure 5.13, Figure 5.14 and Figure 5.15 for Geosynthetics placed at the embankment and ground interface, 100mm above ground and 200mm above ground respectively.

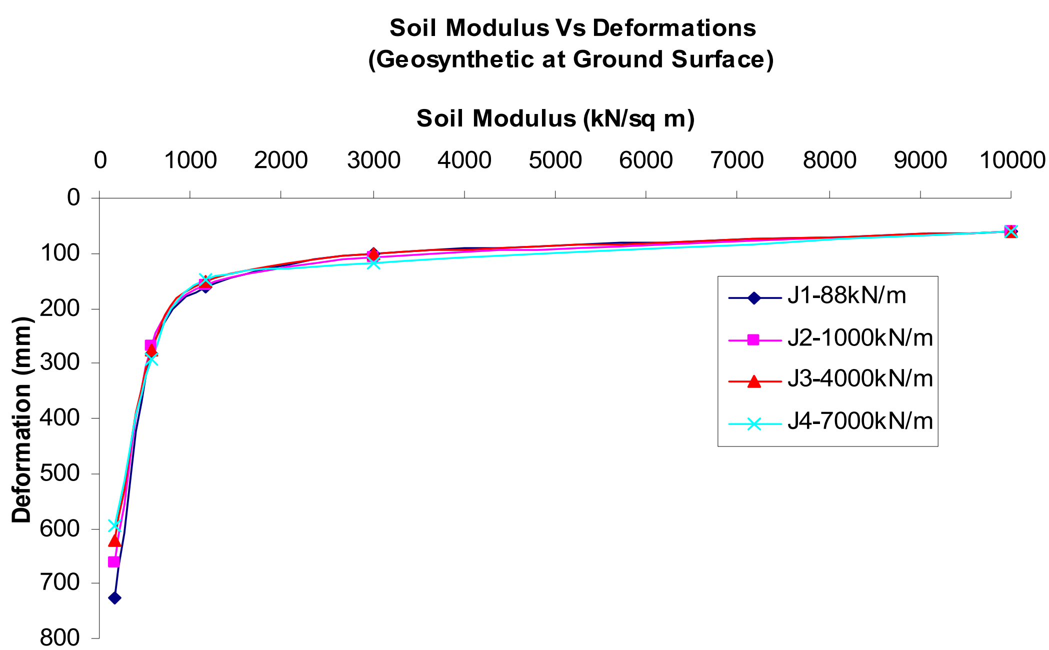


Figure 5.13 Deformation Analyses of Geosynthetics at Ground Surface

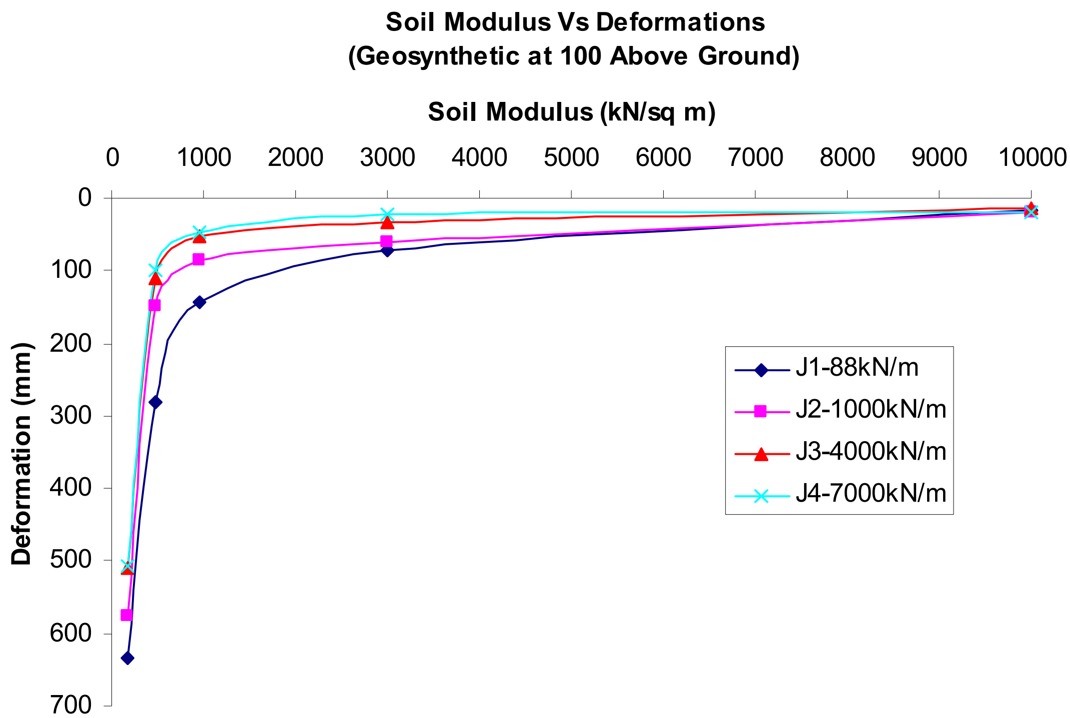


Figure 5.14 Deformation Analyses of Geosynthetics at 100mm above Ground Surface

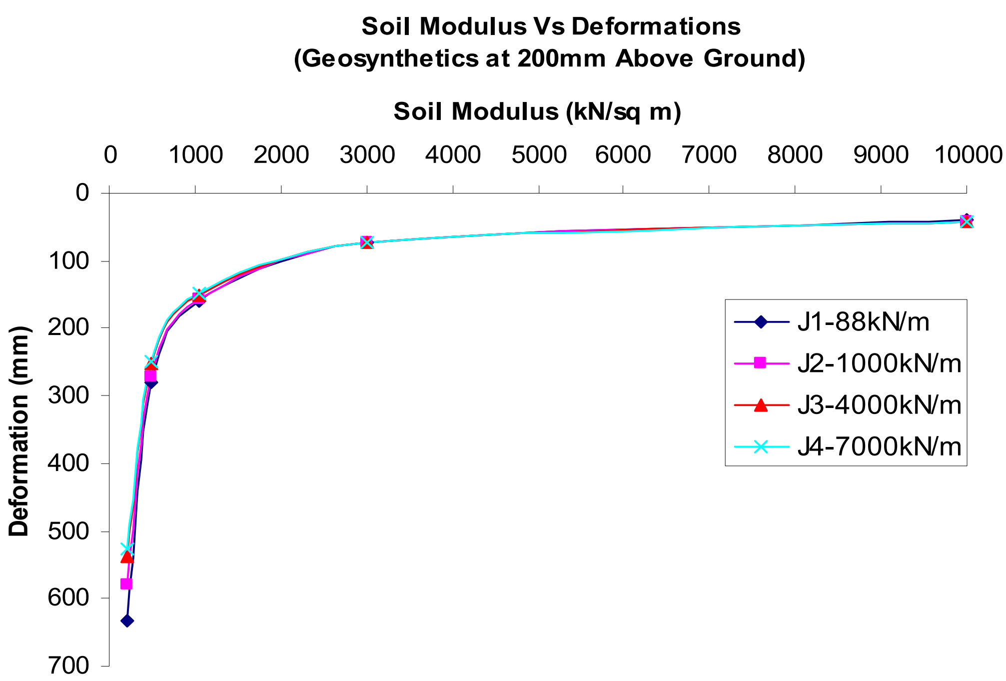


Figure 5.15 Deformation Analyses of Geosynthetics at 200mm above Ground Surface

PLAXIS analysis to study the effects of modified ground conditions in conjunction with varying pile systems in addition to geosynthetic (88 kN/m, 1000 kN/m, 4000 kN/m, 7000 kN/m) have been performed. The results for different combinations of geosynthetic reinforcement and various pile system is illustrated.

The effects of soil stiffness (modulus), in addition to placement of geosynthetics are observed in the current analysis. The stiff soil facilitates the stabilization process and reduces model deformation. It can be inferred from the FEM plots that the effects on deformation is evident for soil modulus less than 3000 kN/m^2. Furthermore, it can be inferred that the geosynthetic placed at 100mm above ground surface in the embankment is ideal, as there is a considerable reduction in model deformation. It can be inferred from the FEM plots that the deformation is over 600mm for models being analyzed with geosynthetics placed at the ground surface. However, for model with geosynthetics placed at 100mm above ground in the embankment provides deformations of less than 600mm. The deformation curve in response to varying soil modulus provides a gradual decrease (soil modulus more than 3000 kN/m^2) in the case of geosynthetics placed at ground level and in case of geosynthetics at 200mm.

5.2.3.1 Stone Column Analysis

To compare and contrast the model response to various stabilization techniques, deformation analyses for combined stabilization system (geosynthetic reinforcement with various pile system) are performed. The FEM outputs of embankment models with geosynthetics at three elevations for stone column are illustrated in Figure 5.16, Figure

5.17, and Figure 5.18 for pile top, 100mm above pile top and 200mm above pile top in the embankment.

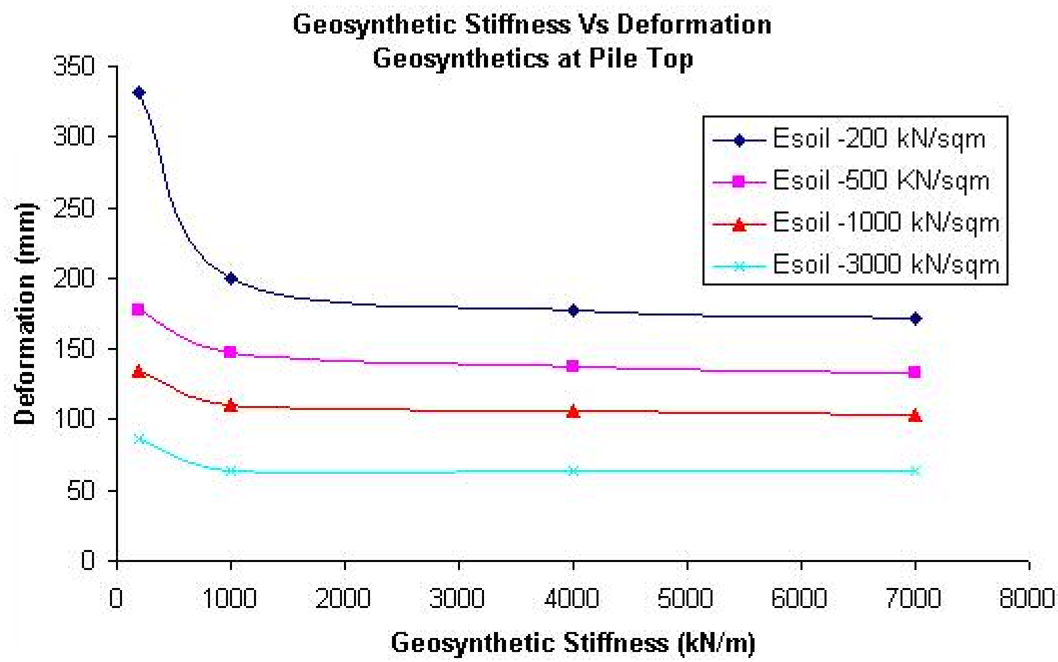


Figure 5.16 Deformation Analyses of Stone Column with Geosynthetics at Ground Surface.

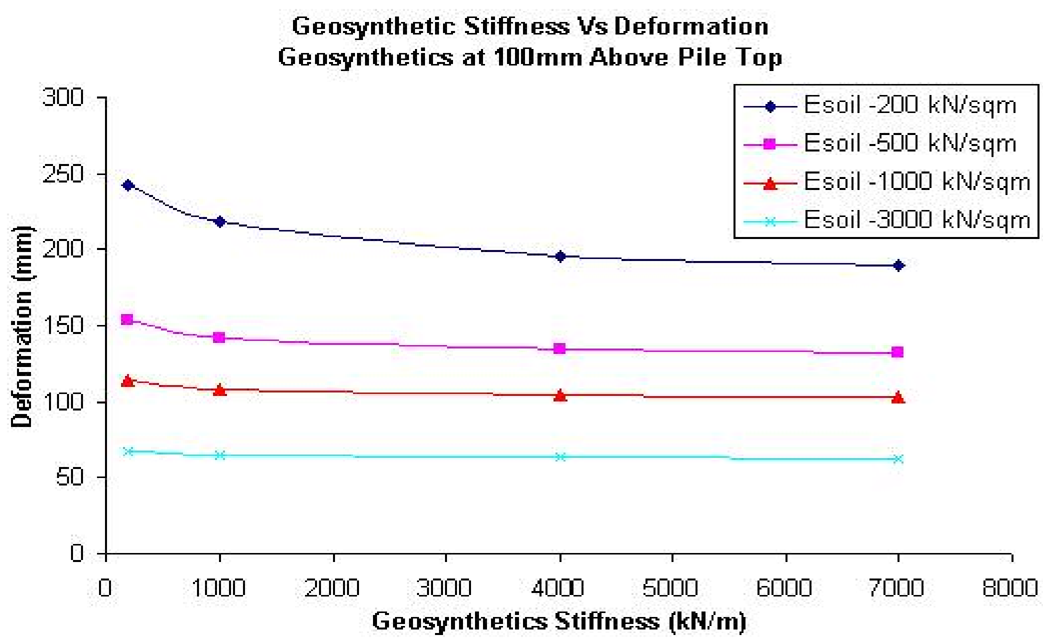


Figure 5.17 Deformation Analyses of Stone Column with Geosynthetics at 100mm Above Ground Surface.

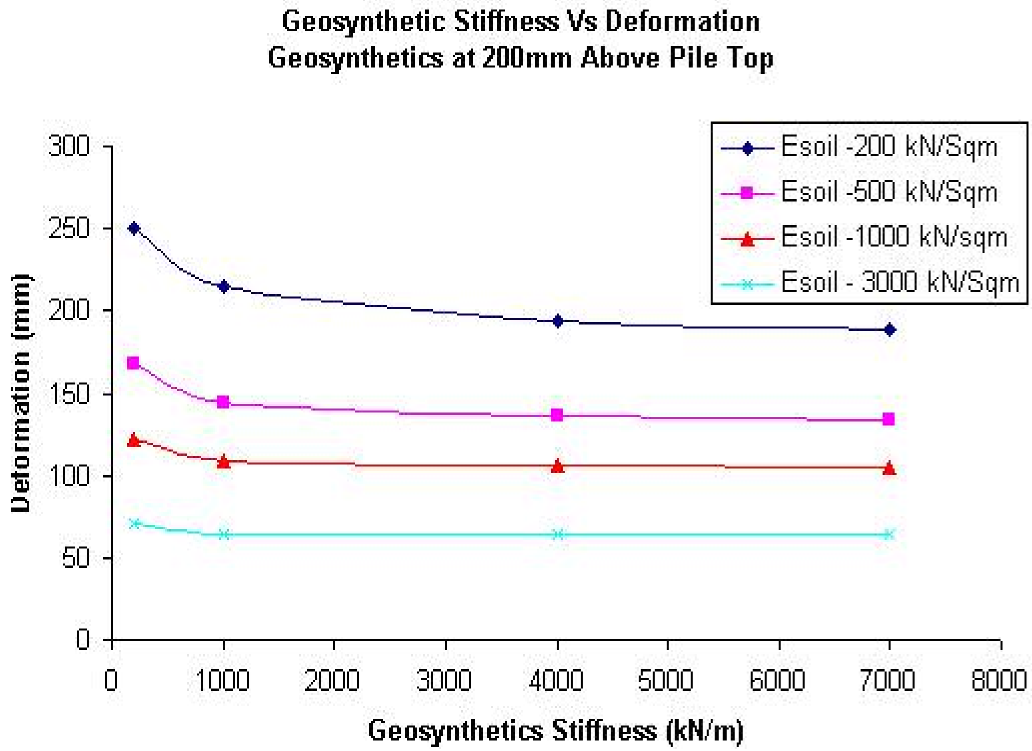


Figure 5.18 Deformation Analyses of Stone Column with Geosynthetics at 200mm above Ground Surface.

The effects of soil stiffness (modulus), in addition to placement of geosynthetics and stone columns are analyzed. Considerable decrease in deformation or improved stabilization was observed when the geosynthetic stiffness is increased for low soil modulus (E_{soil} between 200 kN/m^2 and 1000 kN/m^2). However, the stabilization due increase in soil modulus (E_{soil} > 2000 kN/m^2) in conjunction with stiffer geosynthetics is not significant. Placement effects of geosynthetic are evident and show improved stabilization when geosynthetics are placed at 100mm above ground surface in the embankment.

5.2.3.2 Deep Mix Column Analysis

The FEM outputs of embankment models with geosynthetics at three elevations for deep mix columns are illustrated in Figure 5.19, Figure 5.20, and Figure 5.21.

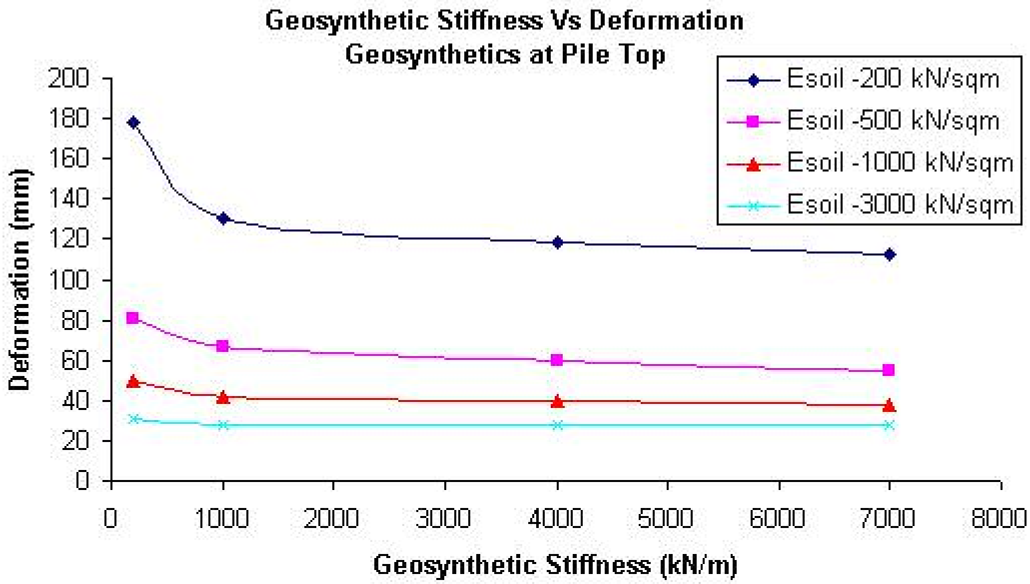


Figure 5.19 Deformation Analyses of Deep Mix Column with Geosynthetics at Ground Surface.

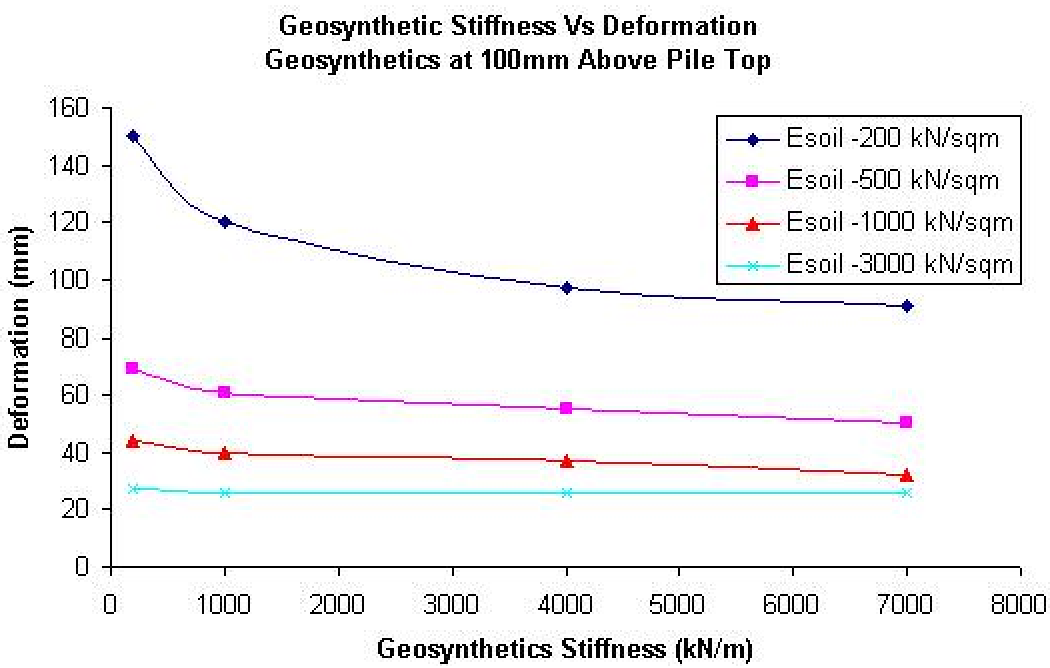


Figure 5.20 Deformation Analyses of Deep Mix Column with Geosynthetics at 100mm above Ground Surface.

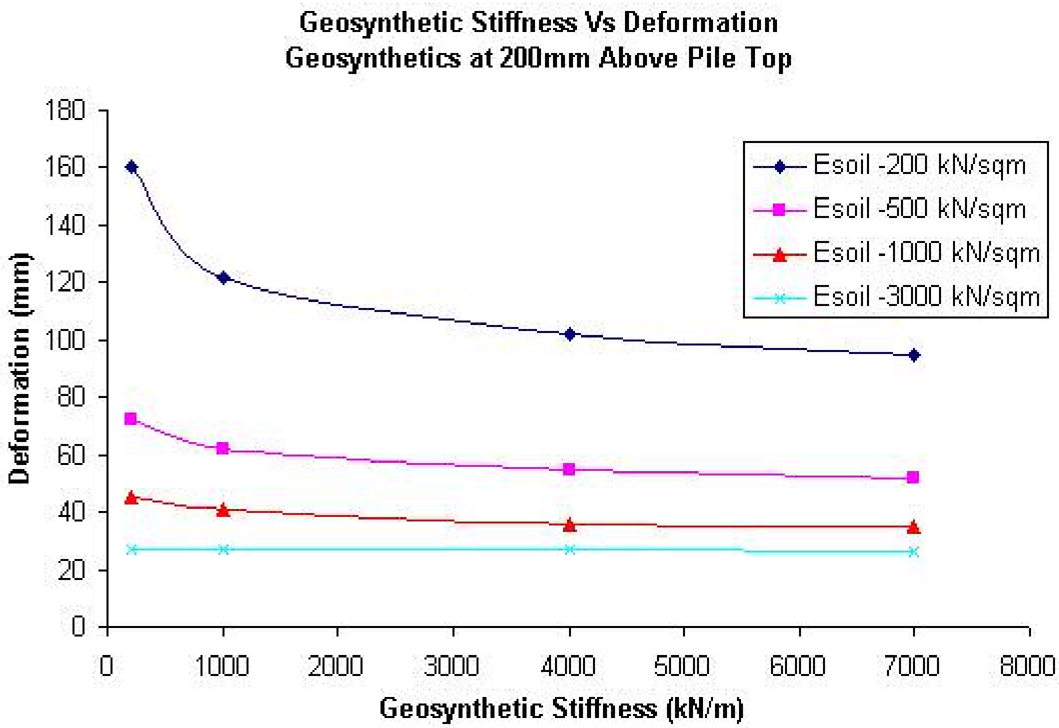


Figure 5.21 Deformation Analyses of Deep Mix Column with Geosynthetics at 200mm above Ground Surface.

The effects of soil stiffness (modulus), in addition to placement of geosynthetics and deep mix columns are analyzed. Considerable decrease in deformation or improved stabilization was observed when the geosynthetic stiffness is increased in case of low soil modulus (E_{soil} between 200 kN/m^2 to 500 kN/m^2). However, the stabilization due increase in soil modulus in conjunction with stiffer geosynthetics is not significant as in the case of stone columns. Placement effects of geosynthetic are evident and show improved stabilization when geosynthetics are placed at 100mm above ground surface in the embankment for low soil modulus. The location effects of geosynthetic are not evident for higher soil modulus.

5.2.3.3 Treated Timber Column Analysis

The FEM outputs of embankment models with geosynthetics at three elevations for treated timber columns are illustrated in Figure 5.22, Figure 5.23, and Figure 5.24.

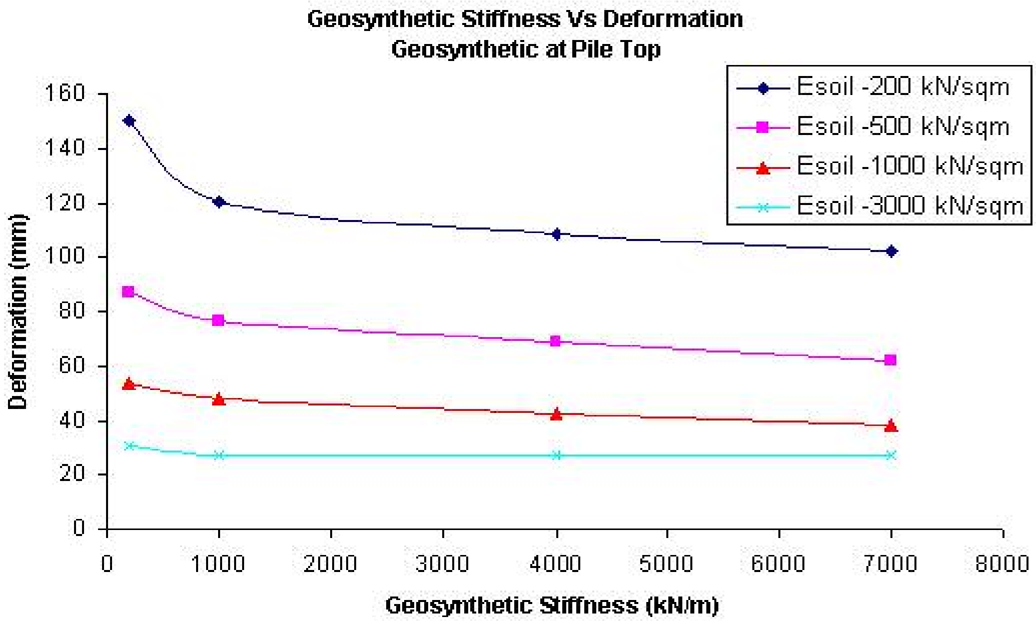


Figure 5.22 Deformation Analyses of Treated Timber Column with Geosynthetics at Ground Surface.

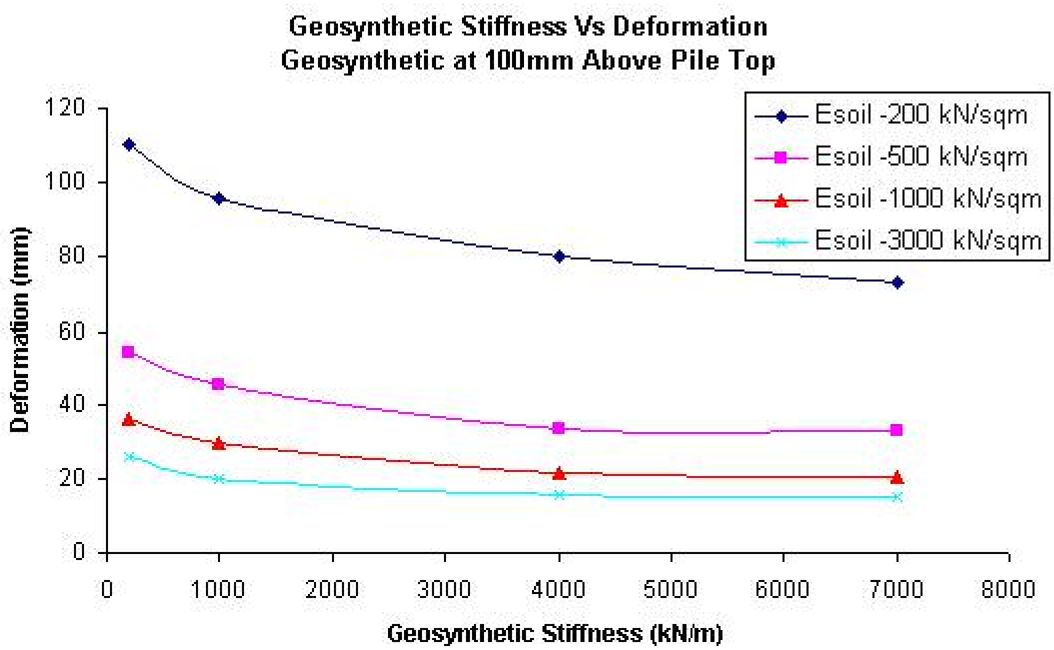


Figure 5.23 Deformation Analyses of Treated Timber Column with Geosynthetics at 100mm above Ground Surface.

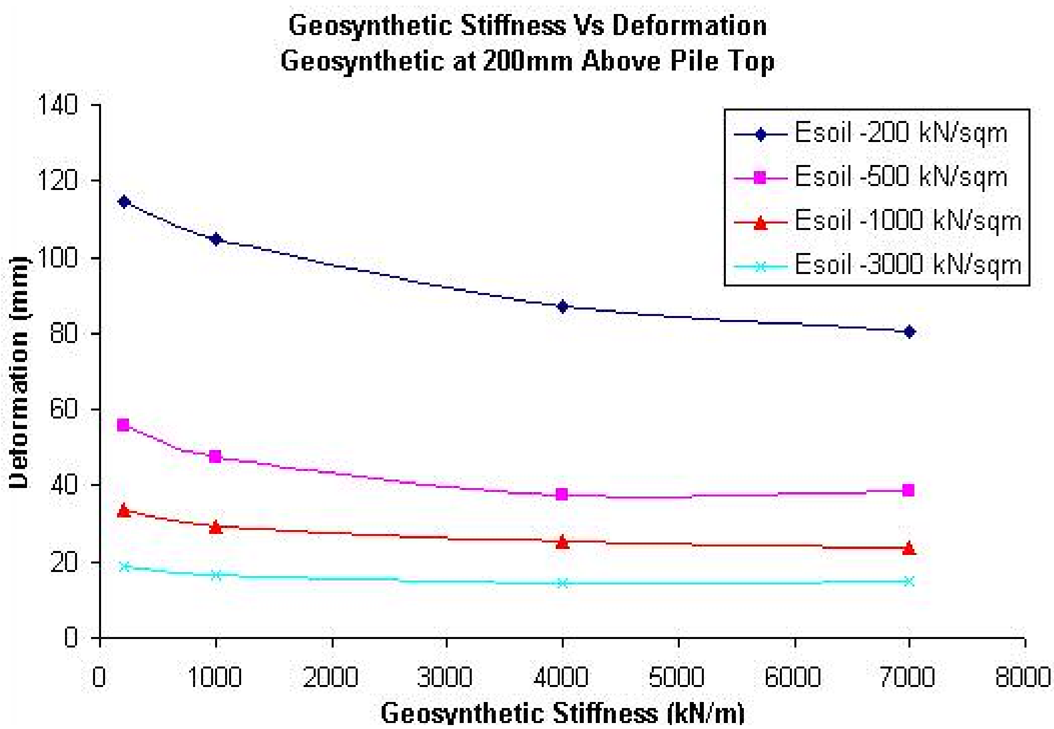


Figure 5.24 Deformation Analyses of Treated Timber Column with Geosynthetics at 200mm above Ground Surface.

The effects of soil stiffness (modulus), in addition to placement of geosynthetics and treated timber columns are analyzed. Considerable decrease in deformation or improved stabilization was observed when the geosynthetic stiffness is increased in case of low soil modulus (E_{soil} between 200 kN/m^2 to 500 kN/m^2). The magnitude of stabilization is evident from the FEM output as represented in Figure 5.22, Figure 5.23, and Figure 5.24. Placement effects of geosynthetic are evident and show improvement in stabilization when geosynthetics are placed at 100mm above ground surface in the embankment for low soil modulus. The location effects of geosynthetic are also evident for higher soil modulus.

5.2.3.4 Concrete Column Analysis

The FEM outputs of embankment models with geosynthetics at three elevations for concrete columns are illustrated in Figure 5.25, Figure 5.26, and Figure 5.27.

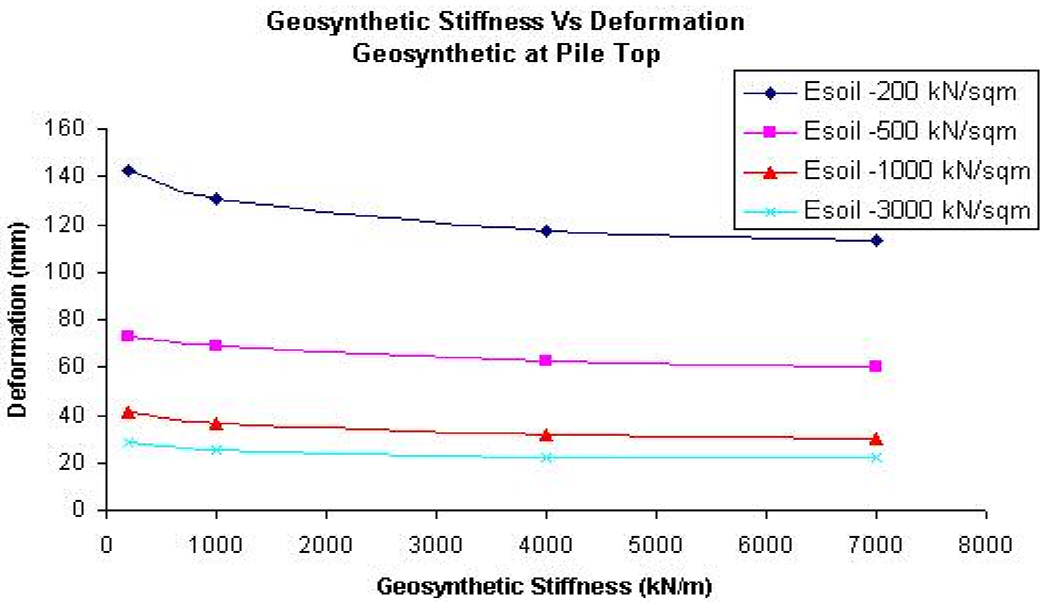


Figure 5.25 Deformation Analyses of Concrete Column with Geosynthetics at Ground Surface.

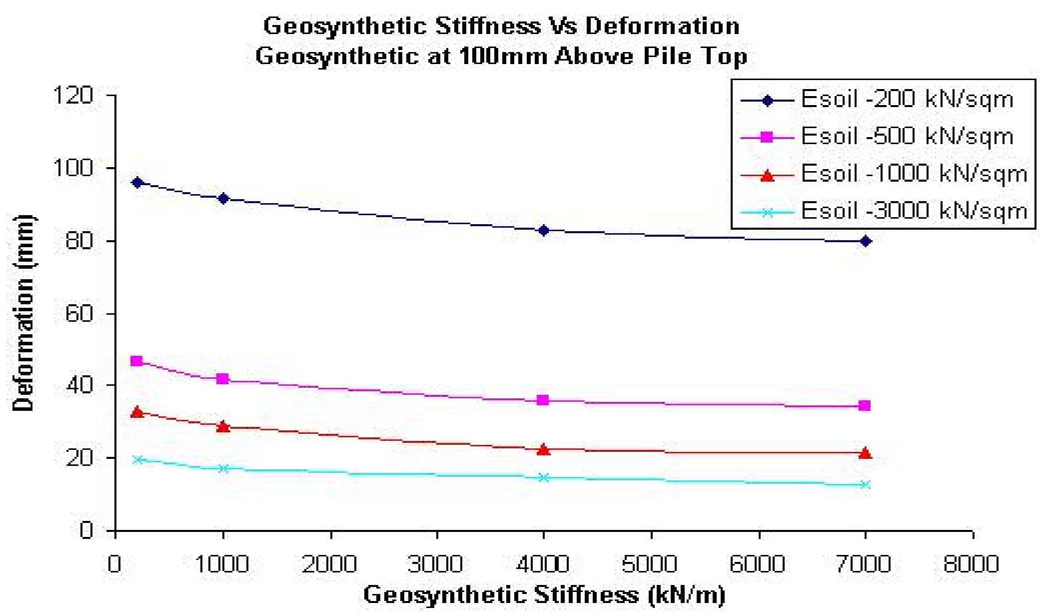


Figure 5.26 Deformation Analyses of Concrete Column with Geosynthetics at 100mm above Ground Surface.

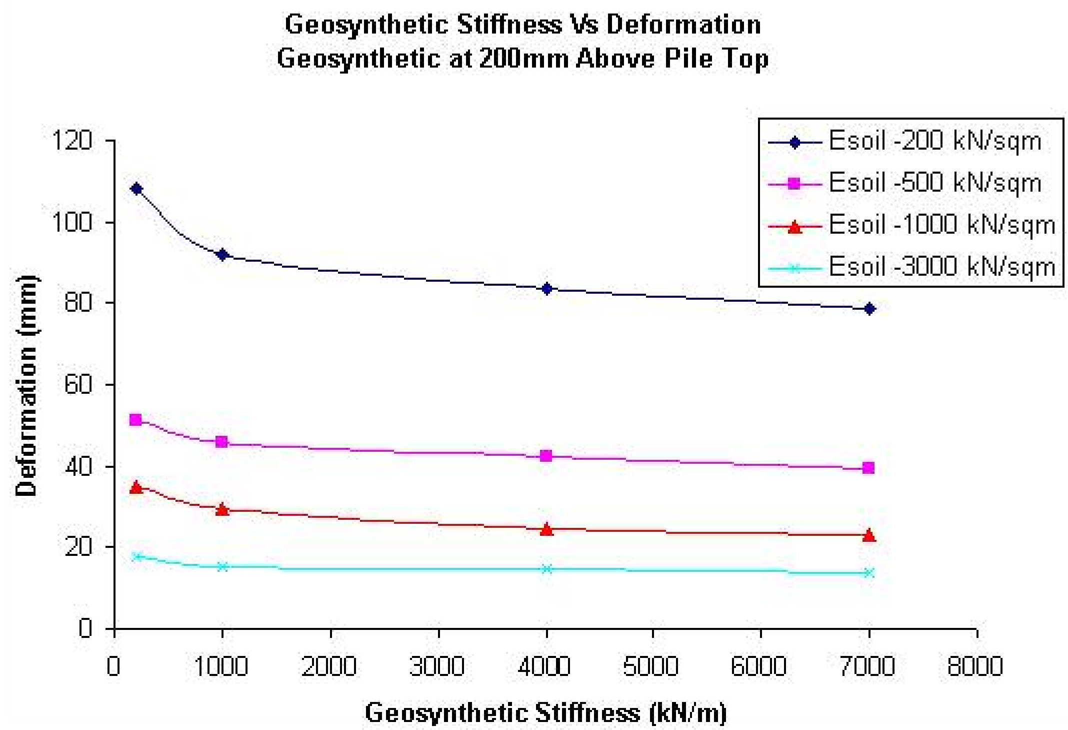


Figure 5.27 Deformation Analyses of Concrete Column with Geosynthetics at 200mm above Ground Surface.

The effects of soil stiffness (modulus), in addition to placement of geosynthetics and concrete columns are analyzed. Significant decrease in deformation or improvement in stabilization was observed when the geosynthetic stiffness is increased in case of low soil modulus (E_{soil} between 200 kN/m^2 to 500 kN/m^2). The magnitude of stabilization is evident from the FEM output as represented in Figure 5.25, Figure 5.26, and Figure 5.27. Placement effects of geosynthetic are evident and show improvement in stabilization when geosynthetics are placed at 100mm above ground surface in the embankment for low soil modulus. The location effects of geosynthetic are also evident for higher soil modulus.

In the FEM model output, with the geosynthetic placed at pile top, it can be observed that the effects of geosynthetic stiffness is evident for stiffness less than 1000 kN/m. However, when the geosynthetics are placed at 100mm and 200mm above pile top in the embankment, the reduction in deformation is evident with the stiffness of up to 2000 kN/m-3000 kN/m. The stabilization process, which helps reducing the deformations, is due to better load transfer from the yielding zone (soft soils) to non-yielding zone (pile head). Furthermore, it can be inferred that the geosynthetic placed at 100mm above ground surface in the embankment is ideal, as there is a considerable reduction in model deformation.

5.2.4 Effects of Embankment Height

Stress Reduction Ratio (SRR) and Soil Arching Ratio, considers factors such as column (pile) diameter, column spacing, column stiffness, embankment fill height, unit weight of fill, friction angle of embankment, geosynthetic stiffness as the main parameters in its computation. In order to analysis the existing model to satisfy the serviceability criteria, embankment response to fill height is analyzed and is illustrated in Figure 5.28.. The computation of SRR and soil arching ratio explains the variation in magnitude of ultimate and differential settlement in a particular model. The effect of embankment fill height is one of the most important criterions to be analyzed prior to any kind of extension in completed project. The reduction in the maximum deformation is evident by including geosynthetics in the embankment. Considerable reduction in deformation is observed when the geosynthetic stiffness is increase. The effects of soil arching ratio being greater explains the phenomenon of reduction in deformation for reinforced section.

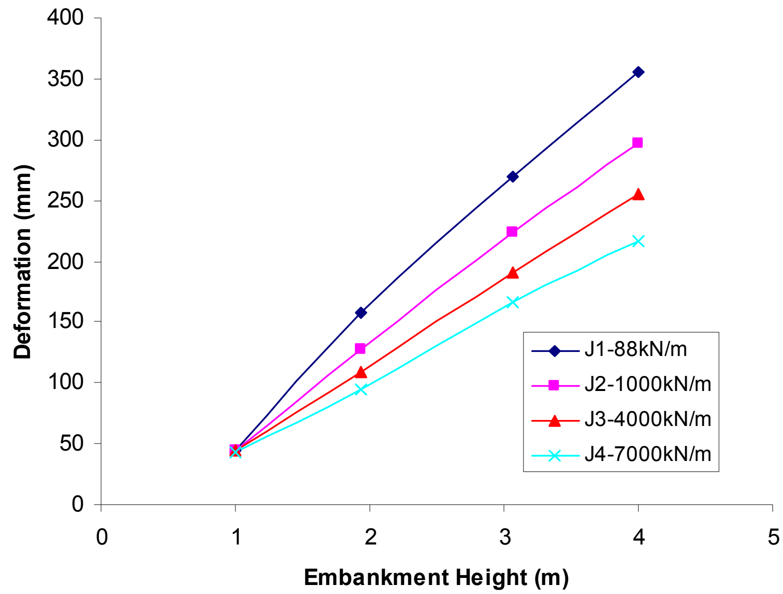


Figure 5.28 Influence of Height of Embankment Fill on Deformation

5.3 Summary

In conclusion, of the discoursed analyses of embankment, it is evident that the ultimate deformation and differential settlement in the present model is a function of elastic modulus of ground and type of ground stabilization (shallow and deep stabilization technique). In the analysis of embankment (a) surface stabilization (b) Different pile systems with and without geosynthetics with varied stiffness (c) Modified soft ground and (d) Embankment Fill height, have been taken into consideration. The result of analysis demonstrates the efficiency of the load transfer mechanism for various treatment methods, which in turn explains the difference in magnitude of ultimate deformations.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary and Conclusions

The Chemico-pile soil improvement method has been used in a full-scale embankment project located at Nong Ngo Hao site near Bangkok. Both laboratory and field investigations were performed to evaluate the effectiveness of lime column on soft soil. Instrumented embankment behavior was observed for 42 days after embankment load was increased to 2m height. Utilizing the improved soils parameters, embankment behavior on improved ground was predicted using the Finite Element Program PLAXIS (Version 8). Further, the embankment behavior was studied with four types of pile systems (Stone Column, Deep Mix Column, Treated Timber Column, and Concrete Columns) along with surface treatment with the geosynthetics. Various stiffness properties of geosynthetics are considered in order to study the deformation of embankment. In addition to reinforcing the embankment with geosynthetic, the location effects of geosynthetic were also studied. The three elevations of geosynthetics, which were considered for the analysis, were (a) at ground surface (b) 100mm above ground surface and (c) 200mm above ground. In order to comprehend the embankment behavior for varying soil conditions, a gamut of undrained shear strength for the top layer of very soft clay were considered in the analysis. The feasibility of future construction of the embankment at Nong Ngo Hao site was also addressed in the present analysis. The following conclusions can be drawn form the current study:

- Reduction in the ultimate deformation of embankment model without surface treatment of up to 67% was observed as the stiffness of pile system was increased. However, reduction in the ultimate deformation of embankment model with surface treatment of up to 86% was observed. It can be inferred that the surface treatment provides a significant contribution in reducing the ultimate deformation.
- It is observed that there is a significant variation in the deformation of embankment with the placement of geosynthetics at different elevations (at ground level, 100mm above pile head and 200mm above pile head). It can be inferred that a considerable amount of reduction in deformation is achieved when the geosynthetics is placed at 100mm above the pile head.
- The reduction in deformation is evident with the increase in stiffness of the pile system for the existing ground conditions. The difference in magnitude of maximum settlement due to low strength geosynthetics to high strength geosynthetics tends to be greater with increase in pile modulus. Increase in lateral stability of piles by increasing the soil modulus would increase the overall improvement of embankment.
- The reduction in deformation is evident with the increase in modulus of the very soft clay. The influence of geosynthetic stiffness is apparent for the soil modulus less than 3000 kN/m^2.

• The reduction in the maximum deformation is evident by including geosynthetics in the embankment.

• The reduction in deformation of about 37% was observed when the geosynthetic reinforcement stiffness is increased from 88 kN/m to 7000 kN/m for a fill height of 4m.

6.2 Recommendations

From the results obtained from the current research, the following recommendations should be considered for future works;

• A method to analyze the multi-layer geosynthetic reinforcement as a beam element needs further research.

• An additional analysis based on varying pile spacing (2d or 4d) would provide a better understanding of embankment model.

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